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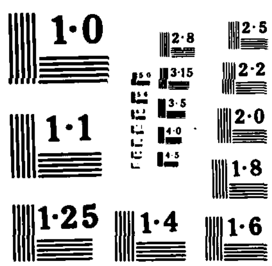
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UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT

NAVAL EDUCATION AND TRAINING CENTER NEWPORT, R.I.

MAY 1981 FPO-1-81 (10)

PERFORMED FOR:

OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
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inshore end of Pier 2. Each facility was inspected by a team of engineer/divers using a combination of visual/tactile and ultrasonic techniques. Critical elements were photo-documented.

Pier 2 appeared to be in generally good condition. The steel pipe piles exhibited some corrosion, but it was not severe. One pile was found to have suffered minor structural damage, probably due to impact. No doncitions were found that would require the pier's strucutral capacity to be downgraded or function to be limited.

The southern section of the quaywall apron exhibited sufficient deterioration and damage to warrant a reduction in its assumed original design live loading of 600 PSF to a loading of 100 PSF. Steel thickness measurements and structural analysis calculations indicate that the capacity of approximately 35% of the steel H-piles has been redured by corrosion to less that the 45 tons of maximum loading per pile required to support the deck live load. Furthermore, approximately 35% of the perimeter piles have been so damaged by impact that they are structurally inadequate to carry the required loads. It is therefore recommended that the loading for the apron be immediately repaired. The remaining piles should be encased in concrete from the pile cap down to elevation -5.0' to protect them from further corrosion.

The northern section of the quaywall apron has veen repaired recently by the placement of concrete jackets from the pile cap down to elevation -5.0'. This has prevented a good structural assessment of the steel H-piles. The remaining metal thickness on the portion of the steel H-pile still exposed is more than sufficient to carry the assumed design deck live load of 600 PSF. However, no conclusion can be made on the overall structural adequacy of these piles. The jackets, being fairly new, were in good condition with one exception. On one pile, the fabric form had ripped so that the concrete jacket can be placed.

FOREWORD

The scope of the inspection at the U.S. Naval Education and Training Center in Newport, Rhode Island and the detail to which it was performed and reported was tailored specifically to the conditions at this facility. This report or the procedure associated with its formation is not intended to be a standard for inspections or reports covering other activities. Attempts are being made, however, toward establishing standards for procedures and formats for inspection and assessment reports. Through these standards, inspections performed by different persons, on many facilities and under a wide range of conditions can be effectively compared. It is expected that the inspections and assessments of the Newport facilities, like previous operations mandated under the underwater portion of the Specialized Inspection Program, will contribute significantly toward achieving that objective.

It should be noted that the choice of the level of inspection and the procedural detail to be employed will be an engineering judgement made separately for each activity/facility to suit its unique situation and needs. Accordingly, the procedures used at the U.S. Naval Education and Training Center in Newport, Rhode Island, rather than serve as a detailed model for inspections elsewhere, will provide guidance with general applicability to future inspections.

EXECUTIVE SUMMARY

The objective of the underwater facility assessments conducted ~~at~~ the U.S. Naval Education and Training Center, Newport, ~~Rhode Island,~~ is to provide a generalized structural condition report of the selected facilities within the activity. The facilities are Pier 2 and the quaywall apron at the inshore end of Pier 2. Each facility was inspected by a team of engineer/ divers using a combination of visual/tactile and ultrasonic techniques. Critical elements were photo-documented.

Pier 2 appeared to be in generally good condition. The steel pipe piles exhibited some corrosion, but it was not severe. One pile was found to have suffered minor structural damage, probably due to impact. No conditions were found that would require the pier's structural capacity to be downgraded or function to be limited.

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The northern section of the quaywall apron has been repaired recently by the placement of concrete jackets from the pile cap down to elevation -5.0'. This has prevented a good structural

assessment of the steel H-piles. The remaining metal thickness on the portion of the steel H-pile still exposed is more than sufficient to carry the assumed design deck live load of 600 PSF. However, no conclusion can be made on the overall structural adequacy of these piles. The jackets, being fairly new, were in good condition with one exception. On one pile, the fabric form had ripped so that the concrete could not be placed. It is recommended that this form be replaced so that the concrete jacket can be placed.

Refer to the following Executive Summary Table for an overview of each facility's construction, recommendations and cost estimates.

U.S. NAVAL EDUCATION AND TRAINING CENTER

NEWPORT, RHODE ISLAND

EXECUTIVE SUMMARY TABLE

Facility	Year Built	No. of Vertical Bearing Piles	No. of Batter Piles	Facility Size	Structure	Recomm
Pier 2	1958	2,796	320	1575'long x 200' wide	14"diameter steel pipe piles	1) Repa Pile with fabr grou 2) Rein in 5-
Quaywall Apron	1957	322 (plus 7 replacement piles)	12	Northern Section: 200'long x 40' wide; Southern Section: 595'long x 40' wide	Steel H-piles: HP 12 x 53 and HP 14 x 73; in the Northern Section, piles concrete-en-cased to El. -5.0'	Southe 1) Rest load to 1 Sect 2a) Rep corr H-pi with post 2b) Pla stru jack corr 3) Plac non-s conc rema prot furth Northe 1) Repl form 15 ar jack

. EDUCATION AND TRAINING CENTER

NEWPORT, RHODE ISLAND

.XECUTIVE SUMMARY TABLE

Batter :s	Facility Size	Structure	Recommendations	Estimated Cost of Recommendations
)	1575'long x 200' wide	14"diameter steel pipe piles	1) Repair hole in Pile B-B, Bent28, with welded wire fabric and epoxy grout.	$1200
			2) Reinspect pier in 5-10 years.	Not applicable
2	Northern Section: 200'long x 40' wide; Southern Section: 595'long x 40' wide	Steel H-piles: HP 12 x 53 and HP 14 x 73; in the Northern Section, piles concrete-encased to El. -5.0'	Southern Section: 1) Restrict live loading on apron to 100PSF (see Section 4.2.4).	Not applicable
			2a) Replace severely corroded steel H-pile sections with new H-pile posts. or	$176,000
			2b) Place 30" diameter structural concrete jackets on severely corroded piles.	$201,000
			3) Place 28" diameter non-structural concrete jackets on remaining piles to protect them from further corrosion.	$194,000
			Northern Section: 1) Replace ripped fabric form on Pile D, Bent 15 and place concrete jacket.	Not applicable

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LIST OF PHOTOGRAPHS

(cont'd)

LIST OF PHOTOGRAPHS

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LIST OF PHOTOGRAPHS

(cont'd)

This report is a product of the Underwater Inspection Program conducted by the Ocean Engineering and Construction Project Office (FPO-1), Chesapeake Division, Naval Facilities Engineering Command (NAVFACENGCOM) under NAVFAC's Specialized Inspection Program. This program sponsors task-oriented engineering services for the inspection, analysis and design and monitoring of repairs for the submerged portions of selected Naval Waterfront Facilities. All services required to produce this report were provided by Childs Engineering Corporation of Medfield, Massachusetts under Task No. 6 of Contract No. N62477-80-C-0102.

The efforts expended and costs required to perform these underwater facility inspections vary greatly with the size, age, kind and construction type of the facilities involved. Other factors peculiar to a particular facility or activity also have an important effect on inspection time and costs. These factors include:

*Type and quantity of biofouling to be cleaned for different levels of scrutiny, both visual and with instruments;

*Tidal range - area exposed at low tide for boat inspections;

*Time and type of last inspection;

*Local environmental factors - salinity, pollution level, temperature, etc., affecting rates of corrosion and marine life;

*Function of the facility and the level of activity associated with that function.

1.1 TASK DESCRIPTION

The scope of work under Task No. 6 of the program required the inspection of the underwater portion of designated facilities located at the Naval Education and Training Center (NETC) in Newport, Rhode Island. The quality of inspection had to be

sufficient to provide an adequate general structural assessment of the facilities and to identify areas of sufficient damage and/or deterioration to warrant immediate repair or a future, more detailed investigation.

1.2 REPORT CONTENT

The report contains a description of inspection procedures, the results of the inspection and analysis of the findings, accompanied by pertinent drawings and photographs. Specifically, the inspection results include a description of the location, construction and function of each facility examined within NETC, its observed condition and a structural assessment of that condition. Recommendations for each facility, including cost estimates (based on present local prices) for any repair work, are also included. Structural assessment calculations and cost estimate breakdowns can be found in the Appendix. Also, as supplementary information, a brief description of NETC is provided to define its location, mission, history, existing facilities, climatological and meteorological data and hydrology.

The purpose of this section is to provide a general description of the Naval Education and Training Center in Newport, Rhode Island (NETC). Included in this section will be brief discussions of NETC's location, mission, history, existing facilities, climatological and meteorological data and hydrology. This information is provided to supplement the later sections of this report and to support all considerations necessary to accurately assess the structural condition of facilities inspected in this survey.

2.1 LOCATION OF ACTIVITY

The Naval Education and Training Center, is located on the southeastern portion of Narragansett Bay, in the county of Newport, approximately 25 miles south of Providence, Rhode Island, at 41° 32' north latitude and 71° 19' west longitude (see Figure 1). Situated on Aquidneck Island, NETC is one of four activities at the Naval Complex which stretches through the towns of Newport, Middletown, and Portsmouth, Rhode Island (see Figure 2).

2.2 MISSION OF ACTIVITY

"The mission of the NETC is "to administer those schools assigned which provide a source from which qualified officers may be prepared for military service; train U.S. Navy enlisted and foreign officer candidates, as required; and provide appropriate logistic support for tenant and support activities".

Additionally, as tasked by the President, Naval War College, Commander NETC, acts as Executive Agent for the area coordination of naval activities in the Narragansett Bay region and as tasked by the Commander in Chief, U.S. Atlantic Fleet, as Senior Officer Present Afloat (SOPA) (administrative) in support of homeported or visiting ships. Commander NETC also serves as the Navy's representative concerning the disposal actions of Navy property declared excess following the 1973 SER [Shore Establishment Realignment Program]."[1]

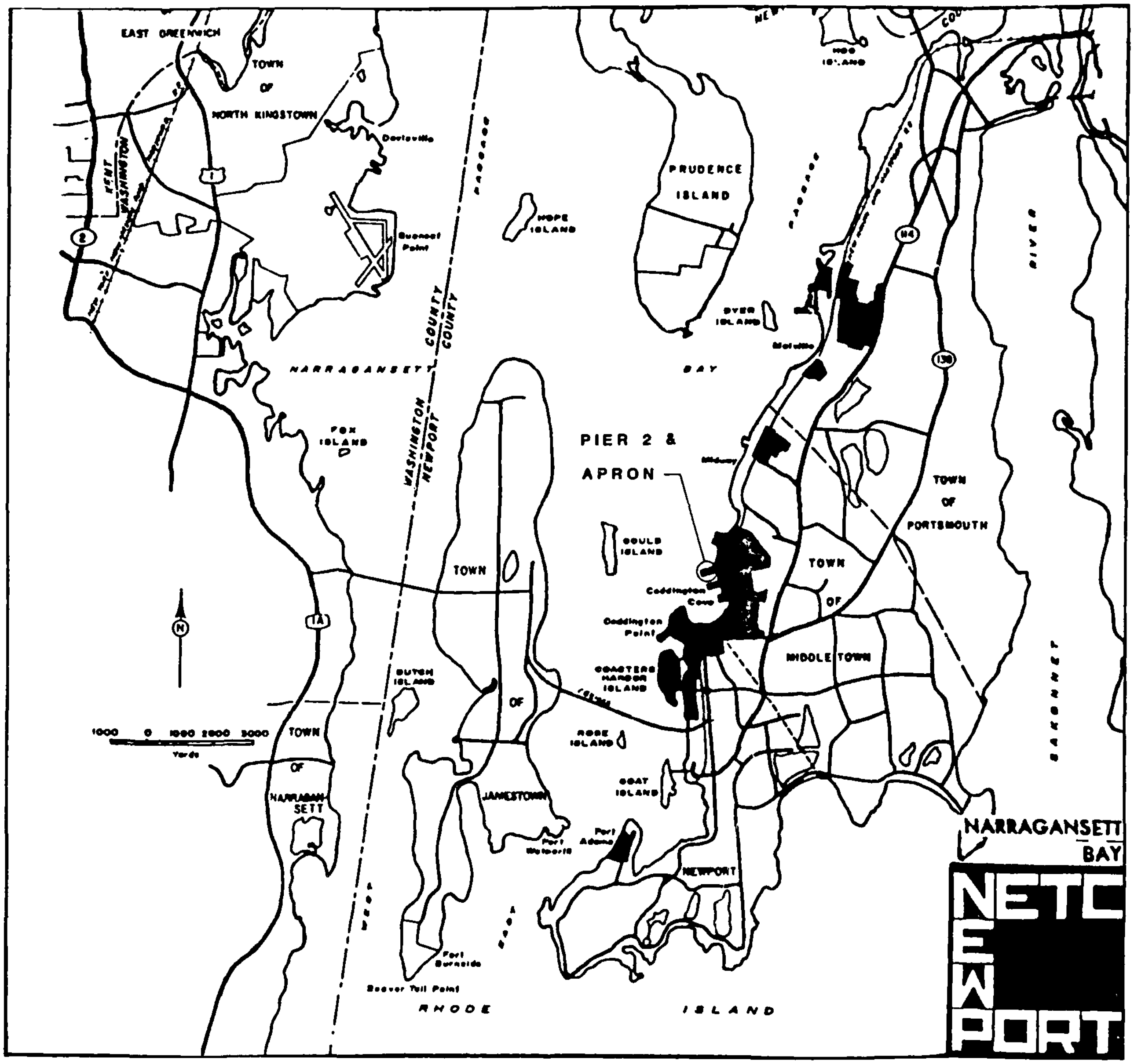
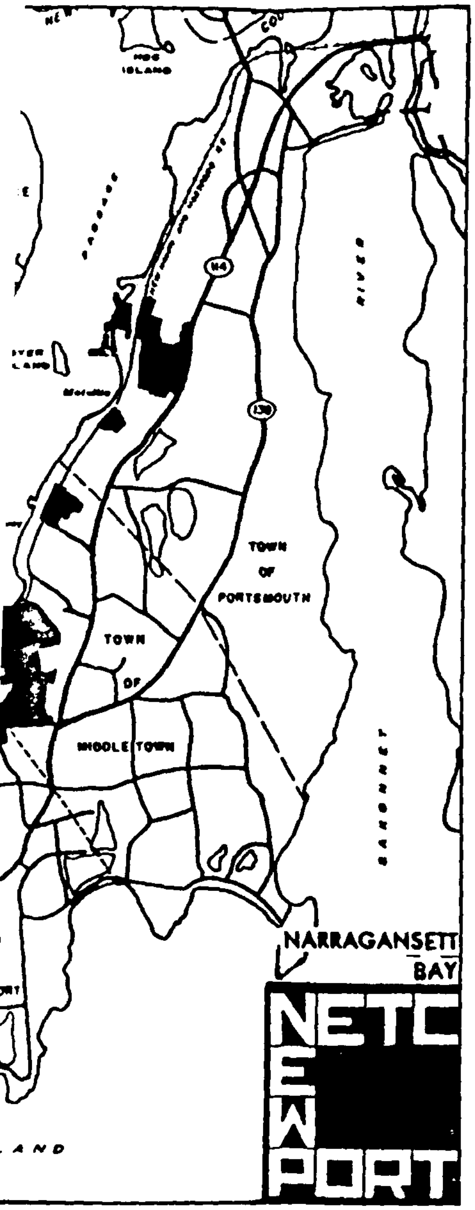


FIGURE 2 LOCUS OF NAVAL FACILITIES ON AQUIDNECK ISLAND
(FROM NETC MASTER PLAN)



AQUIDNECK ISLAND

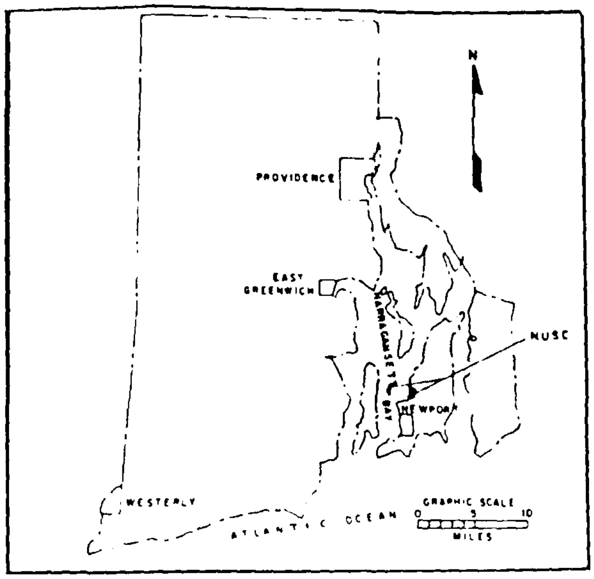


FIGURE 1 LOCUS (FROM NAV. FAC. DRAWING NO. 924671)

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 335 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
AS SHOWN		NETC NEWPORT, RI LOCATION MAPS	FIG NO 1&2

2.3 HISTORY OF ACTIVITY

Ownership of Coasters Harbor Island was taken from the town of Newport by the U.S. Government in 1881, and a shore-based training base was established there soon after. Three years later, in 1884, the Naval War College was founded by Captain Stephen Luce and was established on the island. In 1900, a coal storage facility was added at Melville, and in 1913, the Naval Hospital was established on the mainland. In the ensuing years, with the outbreak of each world war and the Korean conflict, the Naval Complex saw significant increases in size and activity, followed by periods of deactivation and reduction during peacetime.

In 1951 and 1952, the Torpedo Station on Goat Island and the historic Naval Training Station were disestablished respectively. Two years later, in 1954, Hurricane Carol severely damaged much of the facility. Rebuilding efforts after the hurricane included the construction of Pier 1 in 1955 and Pier 2 in 1957. In 1962, Newport became the headquarters for the Commander Cruiser-Destroyer Force Atlantic.

In 1973 the Navy's Shore Establishment Realignment Program (SER) was announced and resulted in the largest single reorganization of naval forces in the Narragansett Bay area. The SER reduced considerably the personnel strength and the physical size of the Naval Complex because of transfers and disestablishments of various commands and activities at Newport and transfer of over 1000 acres of non-essential land and facilities.

"As a result of the April 1973 Shore Establishment Realignment (SER), several commands at the Naval Base, Newport were disestablished and the Naval Education and Training Center Newport RI was established effective 1 April 1974. NETC is, in essence, the result of the combination of five former shore commands; The Naval Base, Naval Station, Naval Officer Training Center, Public Works Center and the Supply Center Annex." [2]

2.4 EXISTING FACILITIES

"The Naval Education Training Center is the largest host activity at the Naval Complex. Established in 1974, following the 1973 SER program, NETC is host for approximately 23 tenants and supported units. Employing over 1,000 civilian personnel and with approximately 140 officers and 280 enlisted personnel assigned to the various departments of the Center, NETC is tasked with the operation of seven schools including the Officer Candidate School (OCS), Officer Indoctrination School (OIS), International Officer Candidate School (INTOCS), Instructor Training School (IT), Communications School, Chaplains School and the Naval Academy Preparatory School (NAPS)."[3]

2.5 CLIMATOLOGICAL AND METEOROLOGICAL DATA

"A combination of phenomena - specifically the Gulfstream, coastal setting, and northern location - make the climate of the Newport area very comfortable. Typically, summers are mild and winters are moderately cold. July temperatures average 72°F with an average range of 63° to 85°F, and January temperatures average 28°F with an average range of 21° to 36°F. The average growing season is 195 days. Relative humidities range from 48 to 84 percent on the average, with the lowest relative humidity occurring in the spring and the highest in the fall.

Average annual precipitation is 42.75 inches falling on approximately 125 days during the year. Annual snowfall averages 37.8 inches. Average wind speed is 10.8 miles per hour and prevails from the northwest in the winter to the southwest in the summer.

Hurricanes are a serious issue in the Newport area. Records indicate that from 1635 to 1965 Rhode Island has experienced or been threatened by hurricane tidal flooding upon 71 occasions. Of these, about 38 caused tidal flooding. The five with the most severe tidal flooding were:

23 September	1815
24 August	1893
21 September	1938
14 September	1944
31 August	1954

The worst storm on record occurred in 1938 when the Bay water height reached 10.8 feet above mean sea level (AMSL) at Newport Harbor. High tide had occurred 45 minutes prior to the storm, but at Providence R.I. and Fall River, MA, high tide and the storm happened simultaneously producing flood surges of 15.7 and 13.7 feet AMSL, respectively. Winds were generally in excess of 75 mph. Gusts in the Providence area hit 125 mph (C.O.E.,1965)".[4]

2.6 HYDROLOGY

"Narragansett Bay, [see Figure 2], consisting of series of submerged river valleys, is approximately 20 miles long and 11 miles wide with a surface area of approximately 102 square miles. The Bay's drainage basins covers about 1,800 square miles in Rhode Island and Massachusetts and provides a fresh water inflow of approximately 1,250 cubic feet per second.

The central portion of the Bay divides into two passages around Conanicut Island (Jamestown) with average depth in the East Passage of 58 feet and 25 feet in the West Passage. A 35 foot (depth) dredged channel connects the passages north of Jamestown.

Narragansett Bay contains over 300 miles of shoreline, largely accounted for by the nearly two dozen islands in the Bay, the largest of which is Aquidneck Island.

The tides in the Bay have a mean range of 3.6 feet at the north of the Bay and 4.6 feet at the head. Tides are semidiurnal."[5]

Tidal ranges for the Naval Complex are as follows:

	Feet
Mean Low Water	0.0
Mean Tide Level	1.8
Mean Tide Range	3.5
Spring Tide Range	4.4

In July 1971, the tidal flow rate at the Jamestown Bridge was measured at 350,000 cubic feet per second. Maximum surface tidal velocities have been measured at 1.5 feet per second (fps) in the West Passage and 2.3 fps in the East Passage. In the vicinity of the Naval Complex, tidal flow currents average around 0.02 fps. Water temperatures range from 31°F. to 75°F.

From May 4-7 and from September 21-22, 1981, a team of engineers and technicians, all certified SCUBA divers, performed on-site underwater inspections of Pier 2 and the quaywall apron at the base of Pier 2, respectively, at NETC, Newport, Rhode Island (see Photo #1). The level of inspection to be performed, the type of structure being inspected, actual on-site conditions and past experience, combined with a thorough knowledge of engineering theory, dictated the inspection procedures that were followed.

3.1 LEVEL OF INSPECTION

The inspection techniques used had to be sufficient to yield information necessary to make a general condition assessment of the supporting structure of each facility, identify any areas that were mechanically damaged or in advanced states of deterioration, and formulate repair and maintenance recommendations and cost estimates. In general, this meant utilizing visual/tactile inspection techniques, accompanied by occasional external measurements employing such instruments as a scale, calipers or ultrasonic steel thickness gauge, where appropriate. Photographic documentation of typical as well as notable or unusual conditions was also obtained.

3.2 INSPECTION PROCEDURE

The scope of work for Task No. 6 of the Underwater Inspection Program required that approximately 1573 lineal feet (LF) of pier and 795 LF of quaywall apron be inspected from the splash zone (practically speaking, the pile cap) to the mudline for general conditions and any gross structural damage or deterioration. The fender and utility systems were beyond the scope of this inspection.

A dive team consisting of two divers and one tender/notekeeper performed the on-site inspection. Past experience has proven this arrangement to be efficient as well as safe.

Photo #1: Looking West from NETC at Pier 2 and the Quaywall Apron.

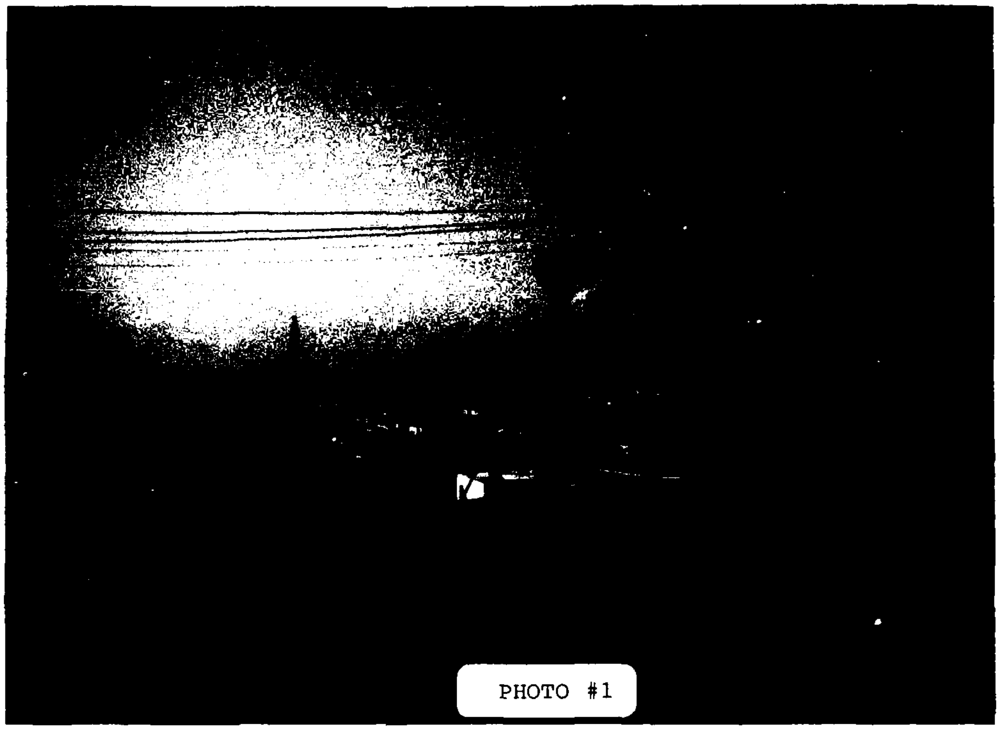


PHOTO #1

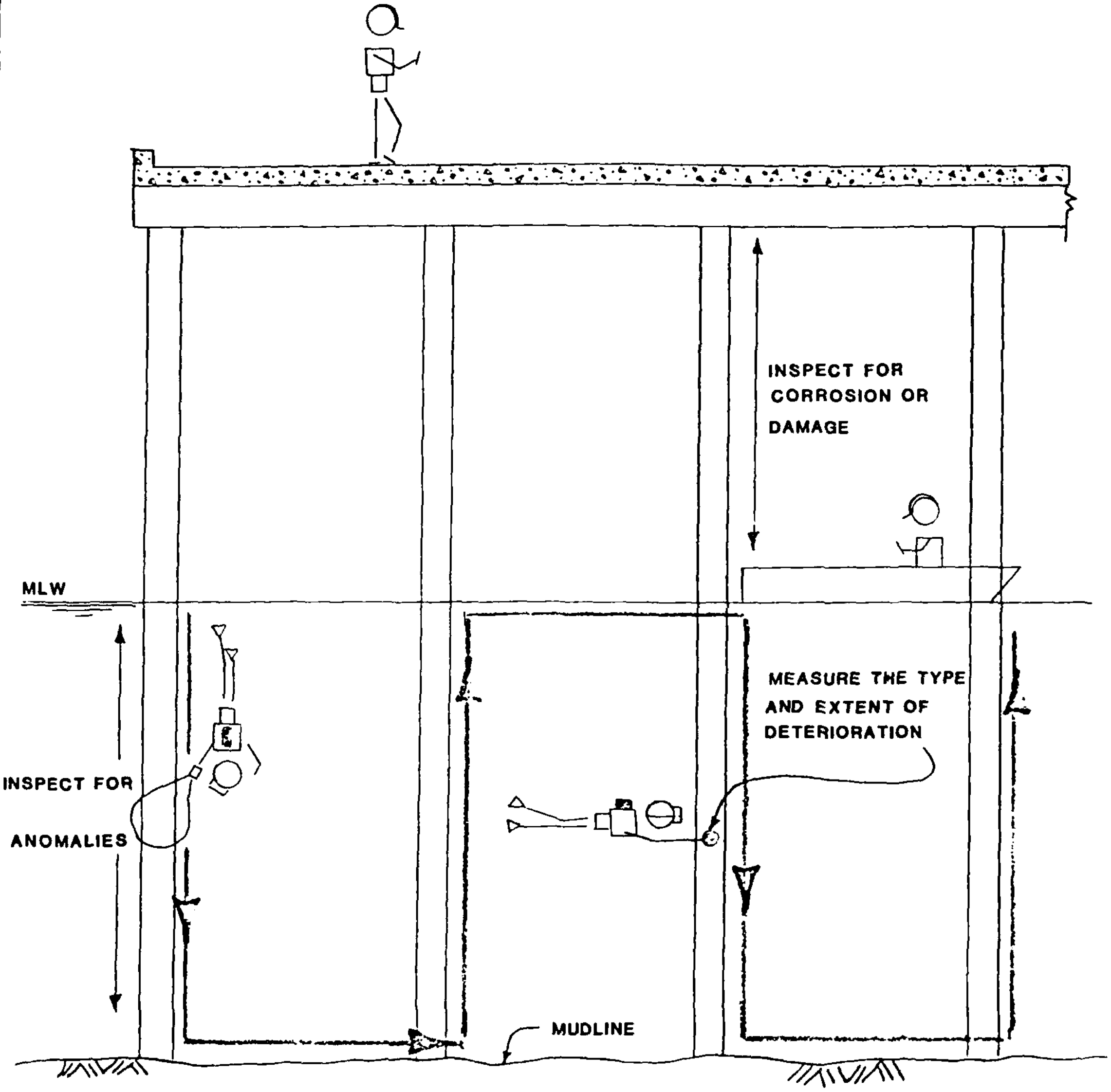
For the Pier 2 inspection, all perimeter piles and every tenth bent were inspected closely in a manner similar to that depicted in Figure 3. Practically speaking, this meant the two divers worked their way down the perimeter piles on one side of the pier from, for example, Bent 10 to Bent 20. They would then traverse the pier while inspecting Bent 20, inspect the perimeter piles on the opposite side of the pier between Bents 10 and 20, then continue along the perimeter piles on this side from Bent 20 to Bent 30 and so on. The remainder of the piles were not inspected (see Y & D Dwg. Nos. 717718 and 717719 in Appendix).

In the case of the quaywall apron, the divers visually inspected all the piles from waterline to mudline. Approximately 8% of the piles, located throughout the facility, were closely inspected (i.e., metal thickness readings were taken; see Figure 7).

Often it was necessary to remove marine growth and/or corrosion from some surface areas of selected piles for an adequate structural assessment. Small patches were frequently cleared during a close inspection.

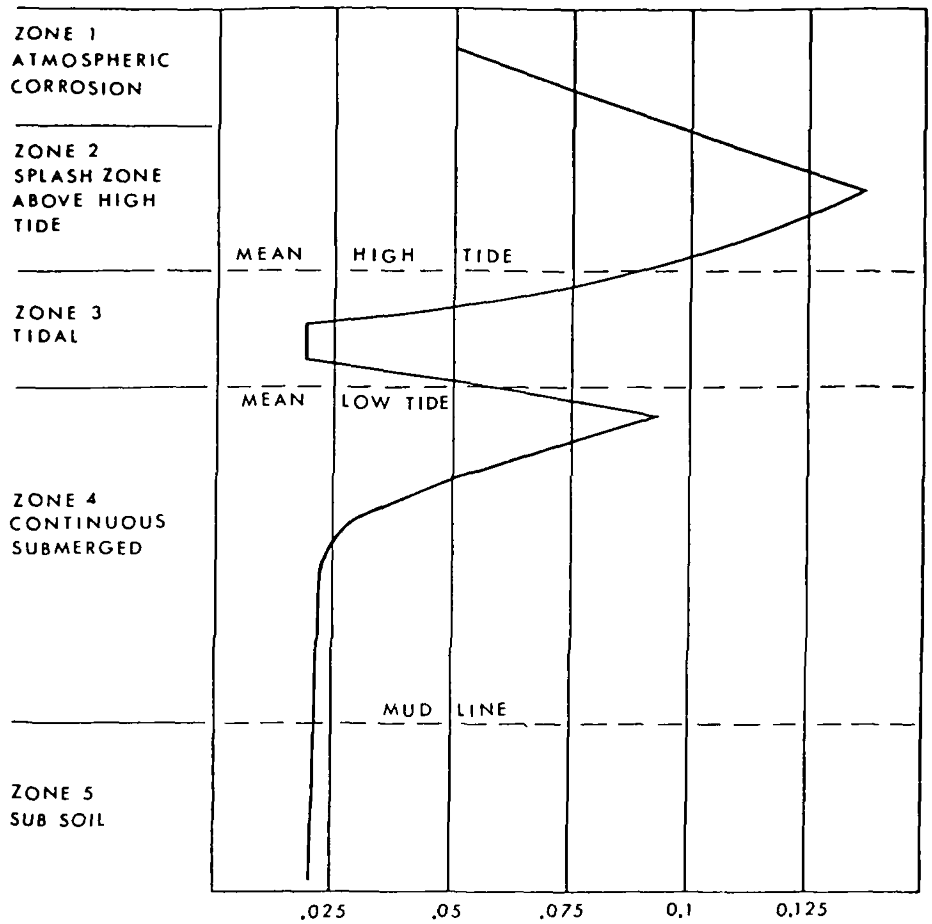
Along with mechanical or impact damage, corrosion of the metal was an important concern. Based on classical corrosion curves, as shown in Figure 4, areas of maximum corrosion usually occur at or around mean low water (MLW), within 2 feet of the mudline, in the splash zone and in areas where a differential oxygen concentration cell is set up. This latter case can occur at the interface or boundary areas between concrete and steel. As a result, the steel adjacent to the concrete is sacrificed to protect the steel under the concrete.

To document the corrosive activity, corrosion profiles were taken on selected piles. Small areas of the pile were cleaned to bare metal at selected elevations (see Photo #2) and metal thickness was measured with an ultrasonic thickness gauge and/or calipers. These readings and empirical corrosion curves based on them are included in the Appendix.



TYPICAL DIVER INSPECTION PATH

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
N/A		NETC NEWPORT, RI	FIG.NO. 3



RELATIVE LOSS IN METAL THICKNESS

CORROSION PROFILE OF STEEL PILING - FIVE YEARS EXPOSURE IN SEAWATER

FROM: S.C. FYRE, "THE PROTECTION OF PILING" IN DESIGN AND INSTALLATION OF PILE FOUNDATIONS AND CELLULAR STRUCTURES, HSAI-YOUNG FANG AND THOMAS D. DISMUKE, EDS. (PENNSYLVANIA: ENVO PUBLISHING CO., INC., 1970), P. 197.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
N/A		CORROSION PROFILE FOR STEEL PILES	FIG NO 4

Photo #2: Typical Ultrasonic Thickness Measurement Station Around El. -5.0' on a Steel Pipe Pile in Pier 2. Note the Welded Splice Connection in the Cleaned Area.

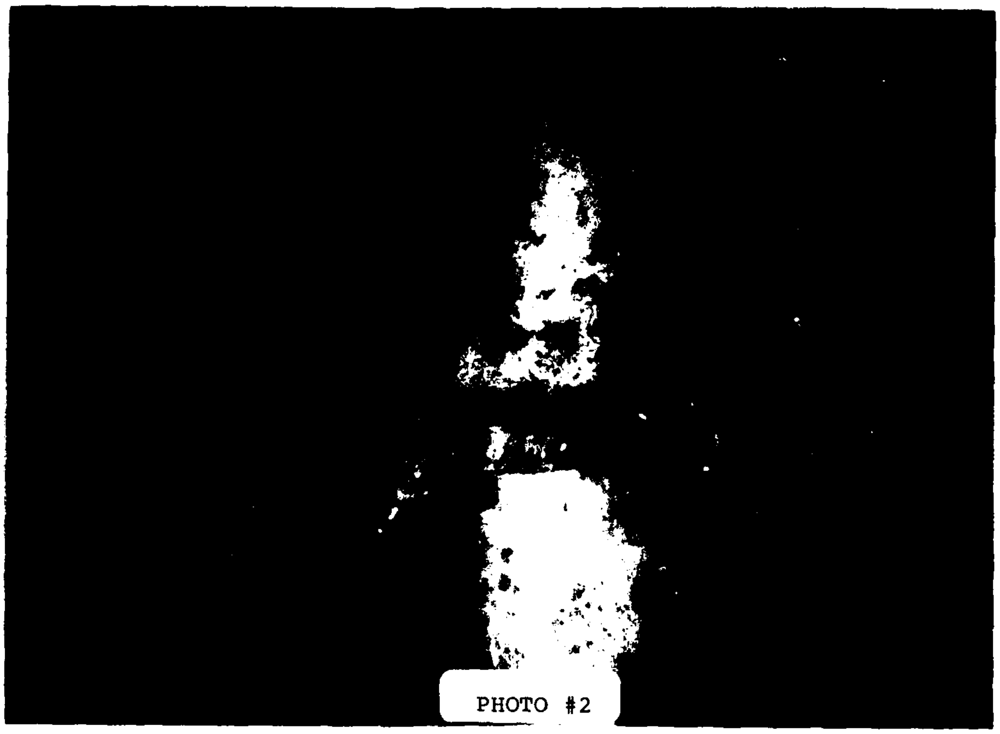


PHOTO #2

It should be noted that during our investigation no destructive testing was carried out. The conditions noted reflect direct observation or measurement of structural components which were accessible. Information which may infer knowledge of conditions of hidden components are based on government-furnished documents, our knowledge of structures in similar environments and/or generally accepted engineering theories.

3.3 INSPECTION EQUIPMENT

Equipment used for the inspection included a Krautkramer D-meter ultrasonic steel thickness gauge with DMR probe and 75 feet of cable, a Minolta SRT 200 camera and strobe, a Nikonos III underwater camera and strobe, dive lights, 200-foot fiberglass tape, 100-foot sounding tape, 6-foot folding rules, calipers, chipping hammers and dive knives.

Choice of equipment was made as a result of past experience. Most of the equipment is straightforward, easy to handle, carry and use, and has proven reliable under hard use.

Ultrasonic steel thickness gauging is preferred over other techniques (such as drilling test holes) since it is non-destructive, easy to handle, fast and reasonably accurate.

Within this section of the report, each facility inspected at NETC is referenced separately. The discussion of each facility is presented in four parts: 1) a description of the construction and function of the structure, which is derived both from the on-site inspection and from the referenced government-furnished drawings; 2) an enumeration of general and specific conditions observed during the on-site inspection; 3) a qualitative assessment of the structural condition of the facility based on the inspection data; and 4) recommendations for actions to be taken to insure long-term, cost-effective maintenance and utilization of the facility. Detailed breakdowns of cost estimates are included in the Appendix.

Marine growth profiles were noted for each facility. In both facilities, there was a distinct band of corrosion, appearing as a ring of orange oxidation, 2' to 4' long, centered around mean low water (MLW; see Photo #3). Little to no marine growth was seen in this area. In Pier 2, below the corrosion band, the marine growth consisted of a brown hairlike growth to approximately 5 feet below MLW. Below the hairlike growth, the piles were covered with clumps of mussels and sea anemones, up to 2" thick, which thinned out near the mudline (see Photos #4 and #5). Much of the marine growth in and just below the tidal zone had been stripped off the outboard face of the perimeter piles in areas where the fendering system was missing or damaged. This was due to abrasion from breasting camels which were being used in place of the original fender system. In the quaywall apron, below the corrosion band, all the piles had a covering of brown hairlike growth, up to 1" thick, barnacles and small tubeworms, which thinned out near the mudline (see Photo #6). Perimeter piles had occasional small clumps of mussels and sea anemones (see Photo #7). Above the band of corrosion in both facilities, the marine growth consisted of a scattering of barnacles which ended in the splash zone.

Photo #3: Typical Corrosion Band of Orange Oxidation, Extending from El. -2.0' to El. +2.0' (Pier 2).

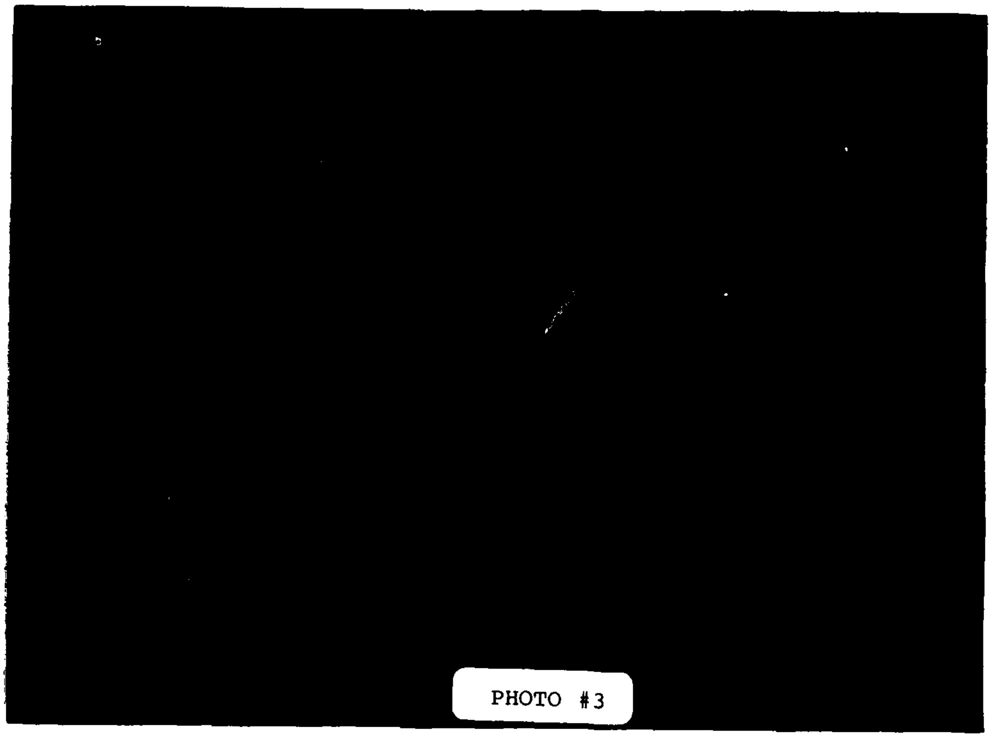


PHOTO #3

Photo #4: Example of Marine Growth Observed at Pier 2 Around El. -5.0'. Note Dense Cover of Sea Anemones and Mussels.

Photo #5: Example of Marine Growth Observed at Pier 2 Around El. -10.0'. Note Patch at Top Cleaned to Bare Metal for Inspection Purposes.

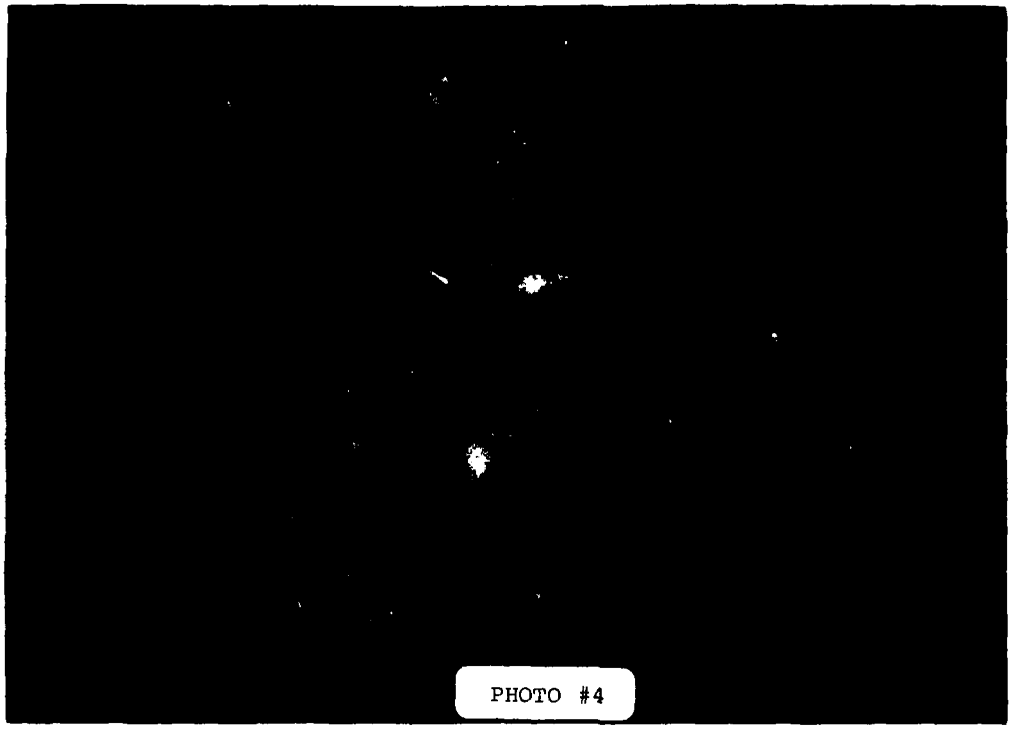


PHOTO #4

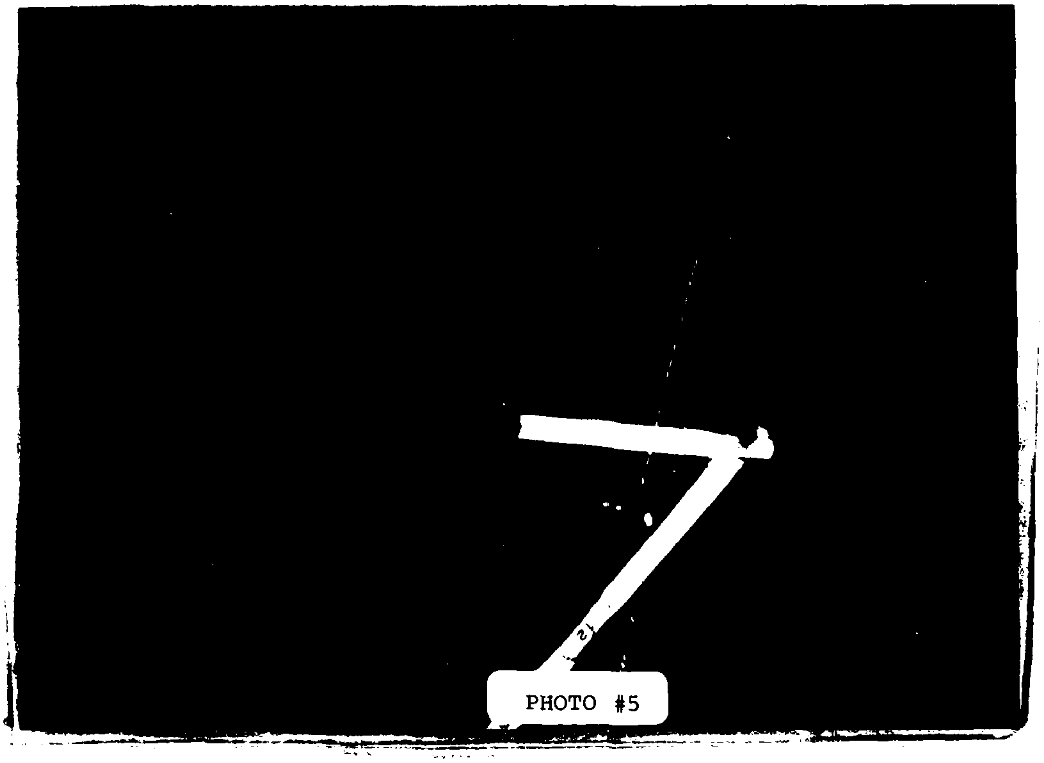


PHOTO #5

Photo #6: Example of Marine Growth Observed in the Quaywall Apron Around El.-20.0'. Note the Areas Cleaned for Ultrasonic Thickness Measurements.

Photo #7: Example of Marine Growth Observed in the Quaywall Apron Around El.-2.0'. Note the Clumps of Mussels Below the Folding Rule.

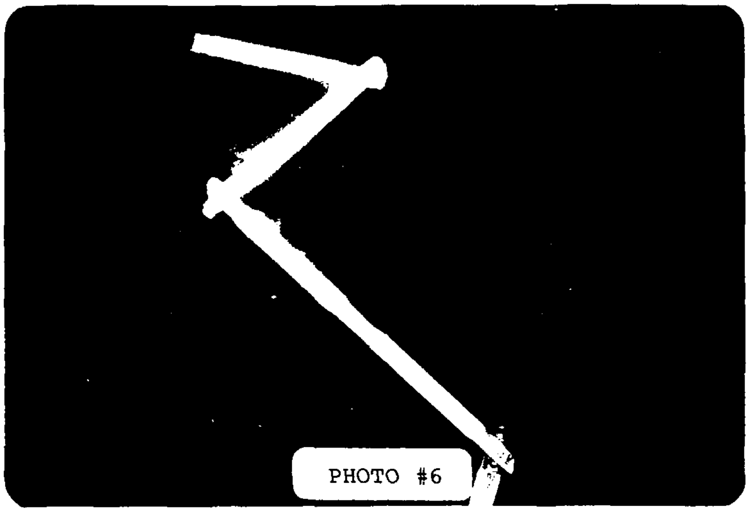


PHOTO #6

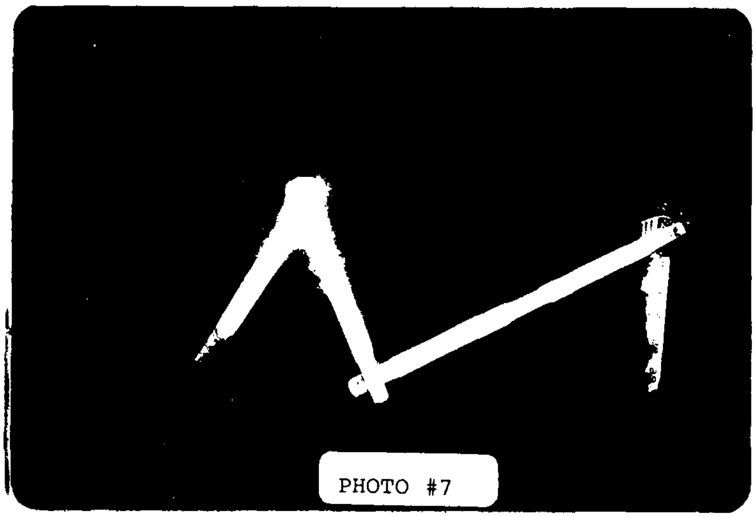
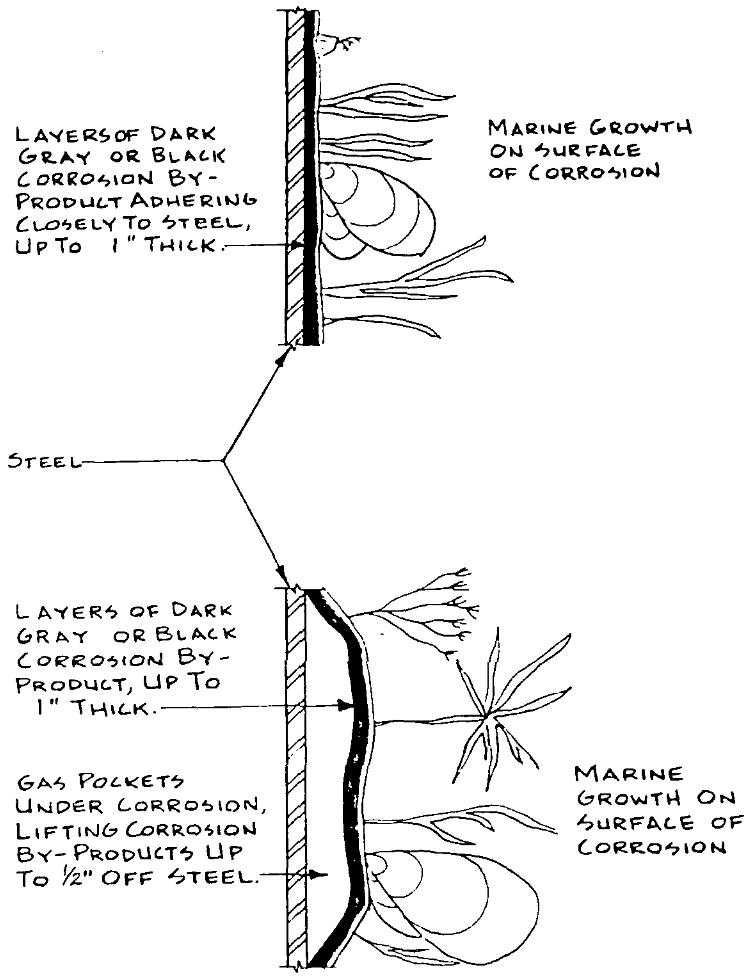


PHOTO #7

On the submerged portions of all the steel piles, deposits of black corrosion by-product with gas pockets trapped beneath were common. At both facilities, this corrosion buildup was heavy, sometimes up to 1" thick. An example of this type of corrosion is illustrated in Figure 5. Under the corrosion buildup, the metal of the steel pipe piles at Pier 2 was basically smooth and unpitted. The metal of the steel H-piles of the quaywall apron, however, was heavily pitted, with pits reaching 1" in diameter and 1/8" in depth.

Corrosion in the splash zone of Pier 2 was light to moderate, and the coating was still apparent in many places. In the quaywall apron, splash zone corrosion was moderate to heavy, and what coating remained was of little to no value.



GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON DC	
N/A		NETC NEWPORT, RI TYPICAL CORROSION BUILDUP	FIG. NO. 5

4.1 PIER 2

4.1.1 Description

Pier 2 is the larger of the two destroyer piers located in Coddington Cove in the town of Middletown, Rhode Island and is situated just north of destroyer Pier 1. Presently, Pier 2 is handling all Navy operations. Building 68, located on the pier, is being renovated to provide space for Ship Intermediate Maintenance Activity (SIMA) operations. Pier 2 was originally designed to accommodate 36 ships (six berths capable of nesting six ships), including destroyers, destroyer-tenders, fleet tugs and fleet oilers.

Pier 2,constructed in 1958, is 1575' long x 200' wide and provides 3,250' of berthing. The reinforced concrete deck is supported by 158 bents of 14" diameter steel pipe piles. From the pile cap to approximately elevation -5.0', the pipe piles have ½" thick walls and are filled with reinforced concrete (see Photo #8). Around elevation -5.0', there is a welded splice connection, and from this point to the mudline, the pipe piles are unfilled and have 3/8" thick walls (see Photo #9). In all,there are 320 batter and 2796 vertical bearing piles (see Figure 6 and , in the Appendix, Y & D Dwg. Nos. 717718 and 717719). The piles have a design bearing capacity of 60 tons. The deck was designed for a uniform live load of 600 PSF or a wheel loading of H-20-44 with 15% impact or a crane loading of a standard gauge 20-ton truck crane with no impact.

References:

Naval Facilities Engineering Command
MASTER PLAN
Newport Education and Training Center
Newport, Rhode Island

Bureau of Yards and Docks
"Pier No. 2, Bulkhead and Utilities"
Y & D Dwg. Nos. 717716,717718 and 717719

Photo #8: Hole in Pile B-B of Bent 28 in Pier 2, Around El. -2.0', Exposing Concrete and Steel Reinforcing.

Photo #9: Typical Welded Splice Connection Around El. -5.0' on a Steel Pipe Pile in Pier 2.

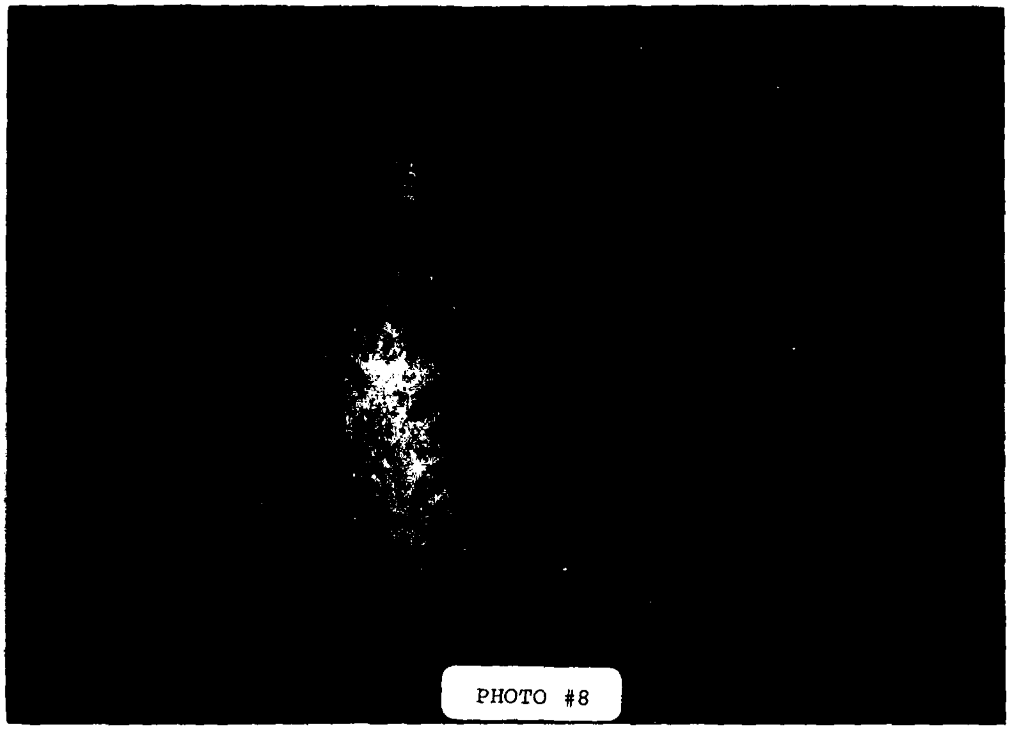


PHOTO #8

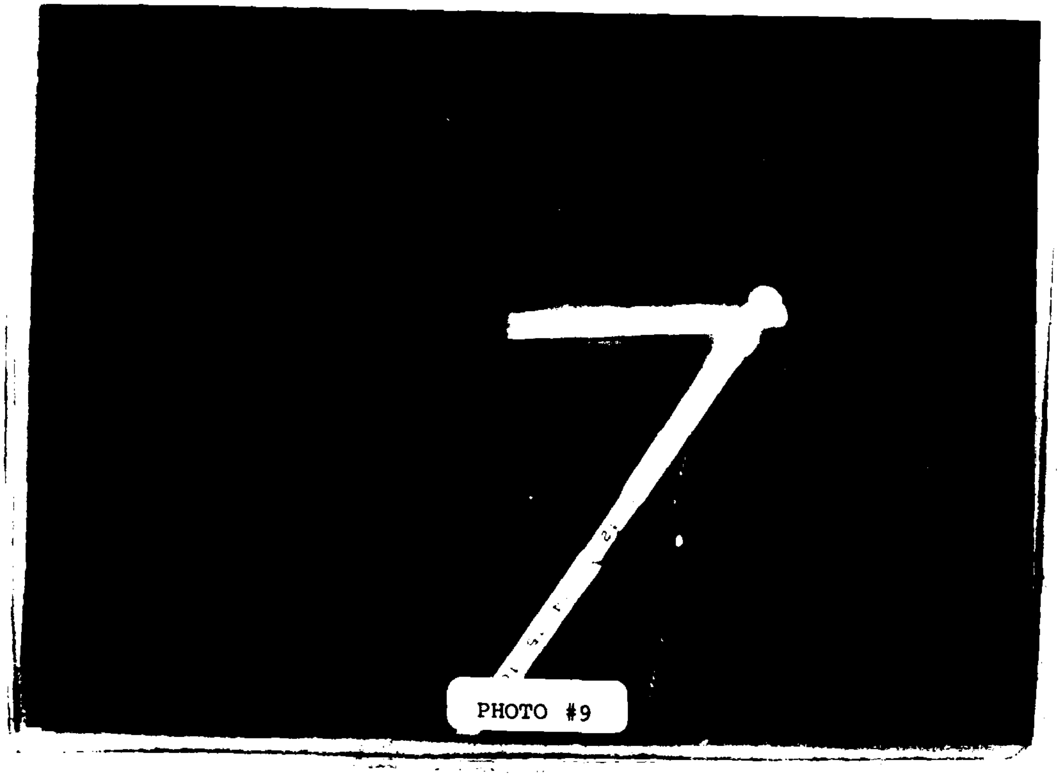
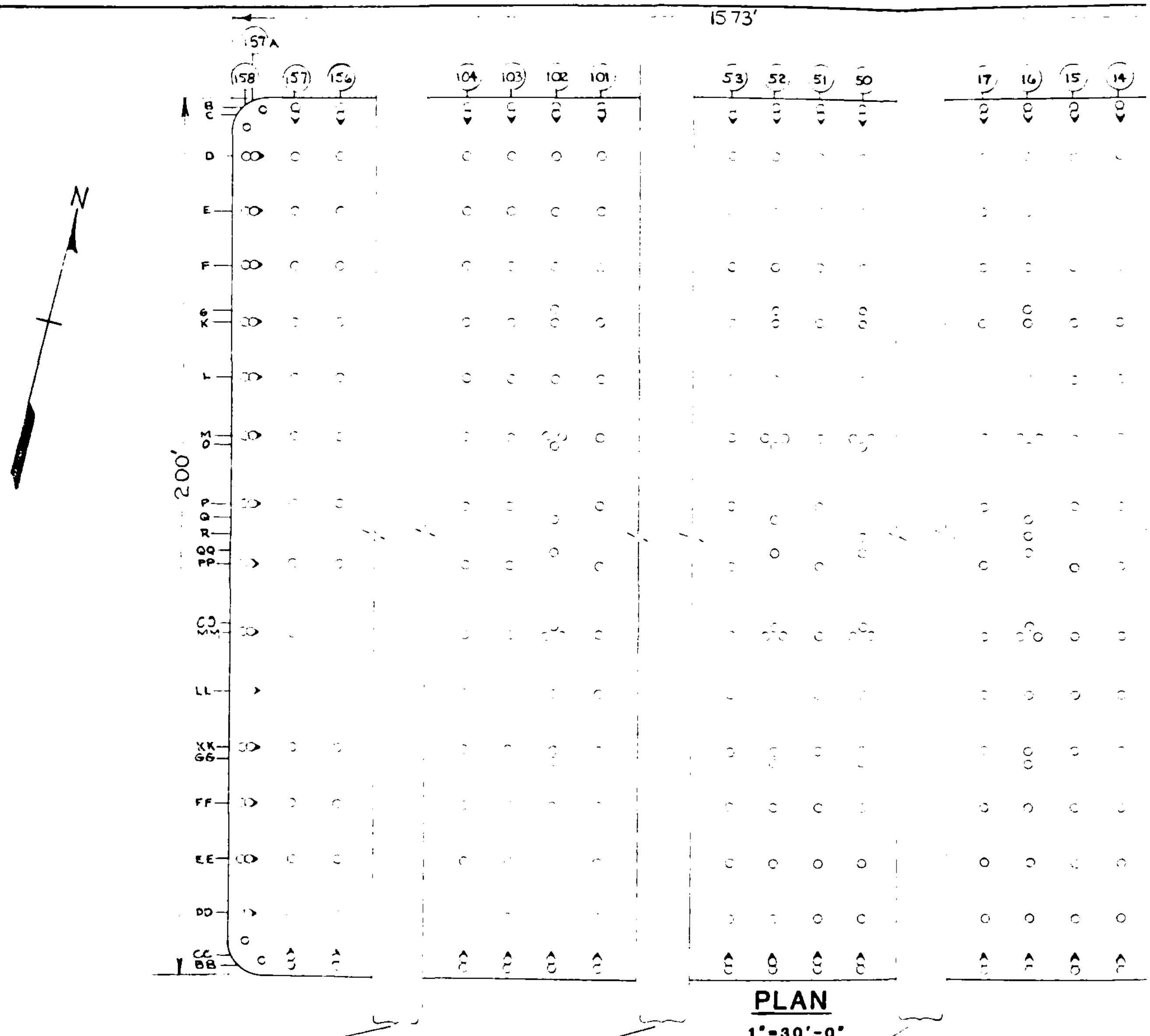


PHOTO #9



PLAN

1" = 30'-0"

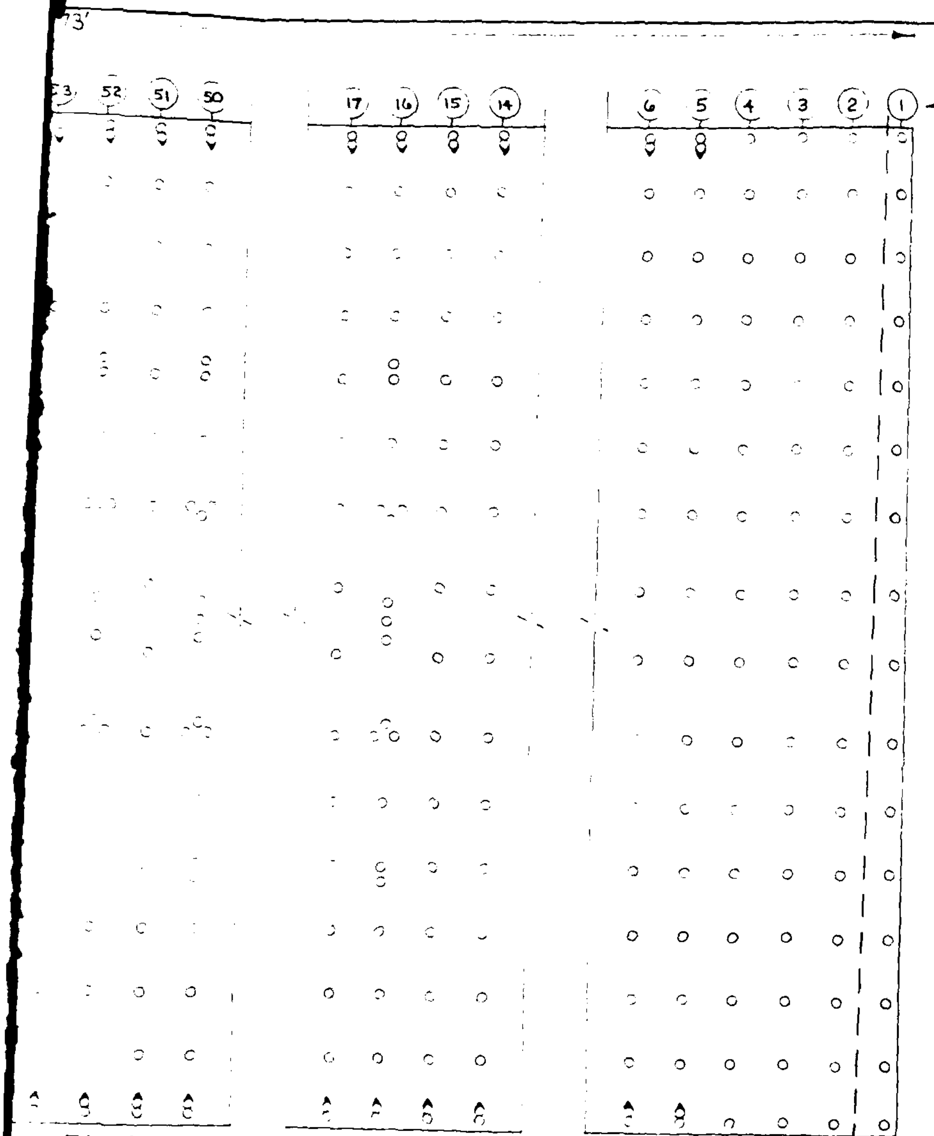
NOTE: BENTS 105 - 155 PILE PLAN FOR ALL BENTS SAME AS BENT #6

NOTE: BENTS 54 - 100 PILE PLAN FOR ODD NUMBERED BENTS SAME AS BENT #6 PILE PLAN FOR EVEN NUMBERED BENTS SAME AS BENT #52

NOTE: BENTS 18 - 49 PILE PLAN FOR ODD NUMBERED BENTS SAME AS BENT #6 PILE PLAN FOR EVEN NUMBERED BENTS SAME AS BENT #16

○ 14" DIA. REINFORCED CONCRETE FILLED PIPE PILE

^ 14" DIA. REINFORCED CONCRETE FILLED BATTERED PIPE PILE



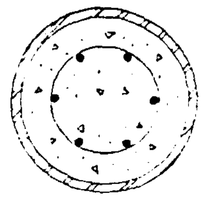
PLAN

1"=30'-0"

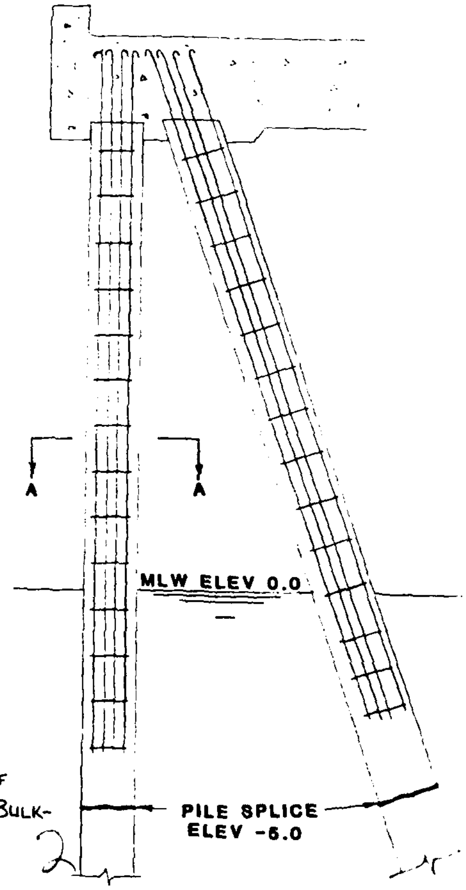
00
R ODD
TS
#6
R EVEN
TS
#52

NOTE: BENTS 18 - 49
PILE PLAN FOR ODD NUMBERED BENTS SAME AS BENT #6
PILE PLAN FOR EVEN NUMBERED BENTS SAME AS BENT #16

NOTE: BENTS 7 - 13
PILE PLAN FOR ALL BENTS SAME AS BENT #6



SECTION A-A



TYPICAL PILE REINFORCEMENT

NOT TO SCALE

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.
0' 10' 20' 40'		NETC NEWPORT, RI — PIER 2 — FIG NO 6

4.1.2 Observed Inspection Condition

The piles of Pier 2 were in fairly uniform condition as described in the introduction to Section 4. The one exception to this was Pile B-B in Bent 28. This pile had a 12" diameter hole on its outboard (south) side centered at elevation -2.0' (see Photo #10,#11 and #12). The steel around the edges of the hole had been corroded to a knife-edge, and the concrete in the vicinity of the hole was missing to half the depth of the pile. One of the reinforcing bars was exposed in this void.

The welded splice connections found were all in good condition (see Photo #13). Most were located around elevation -5.0', but some were found as shallow as MLW and two were found around elevations -10.0' to -15.0'.

Steel thickness readings were taken on two piles from elevation +2.0' or +1.0' down to the mudline. Metal loss above the welded splice connection (½" thick walls) ranged from 0% to 54% with an average of loss of 26% and below the welded splice connection (3/8" thick walls) from 0% to 48% with an average loss of 14% (see Appendix for actual readings).

Photo #10: 12" Diameter Hole Around El. -2.0' on the Outboard Side of Pile B-B, Bent 28, in Pier 2. Note the Void in the Concrete and the Exposed Steel Reinforcing.

Photo #11: Same Description as Photo #10.

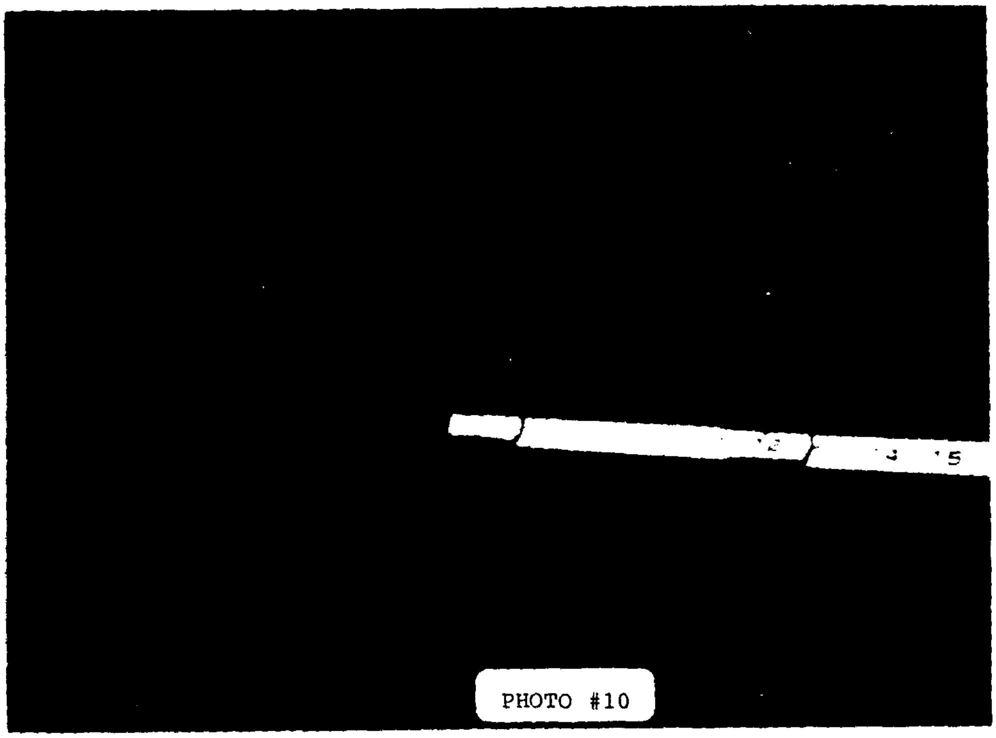


PHOTO #10

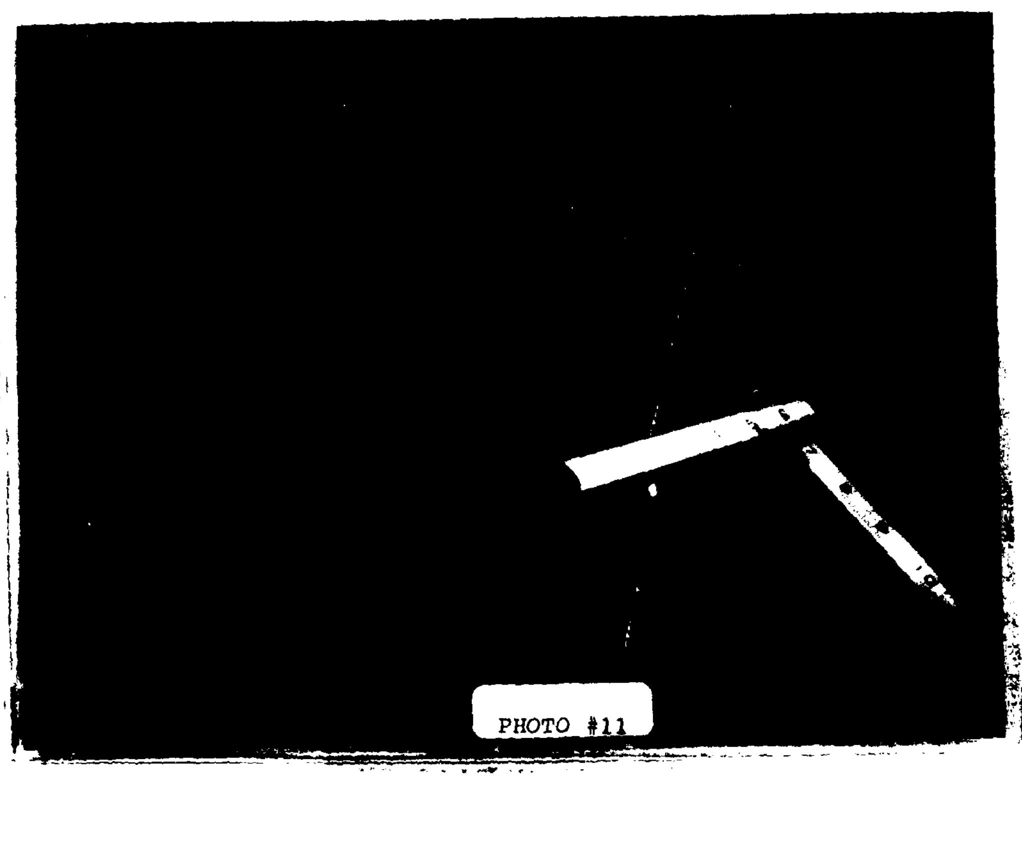


PHOTO #11

Photo #12: View of Top Half of Hole in Pile B-B, Bent 28, in Pier 2. Note How the Edges of the Hole Turn Inward, Indicating Impact Damage.

Photo #13: Typical Welded Splice Connection Around El. -5.0' on a Steel Pipe Pile in Pier 2. Note the Good Condition of the Weld and Adjacent Pipe Pile Sections.

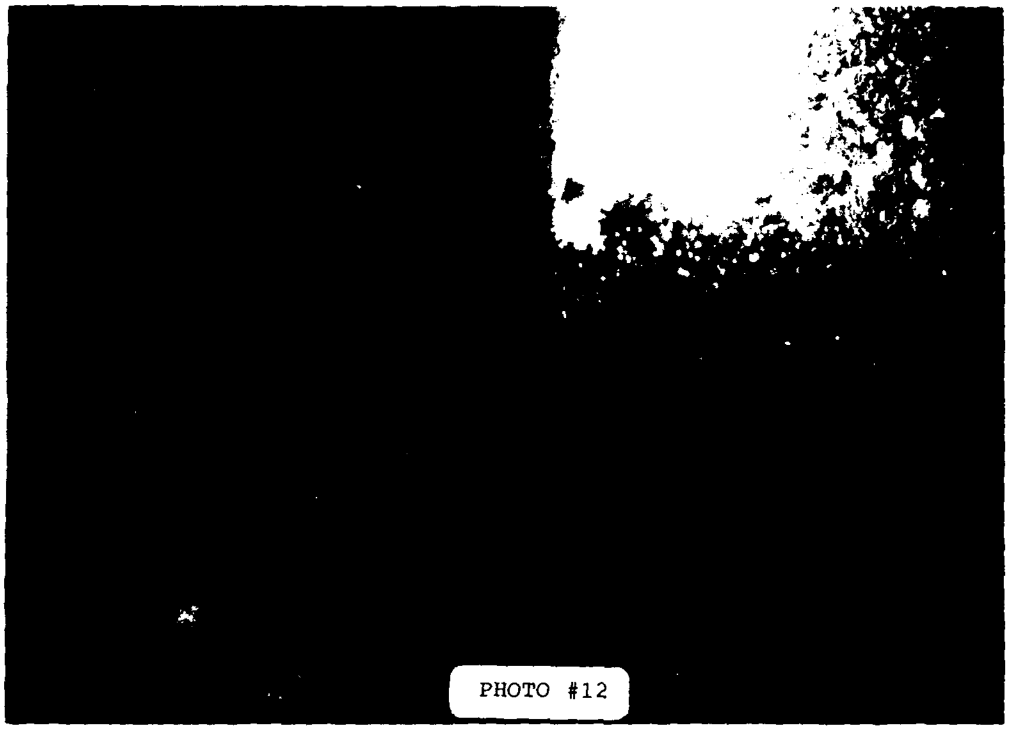


PHOTO #12

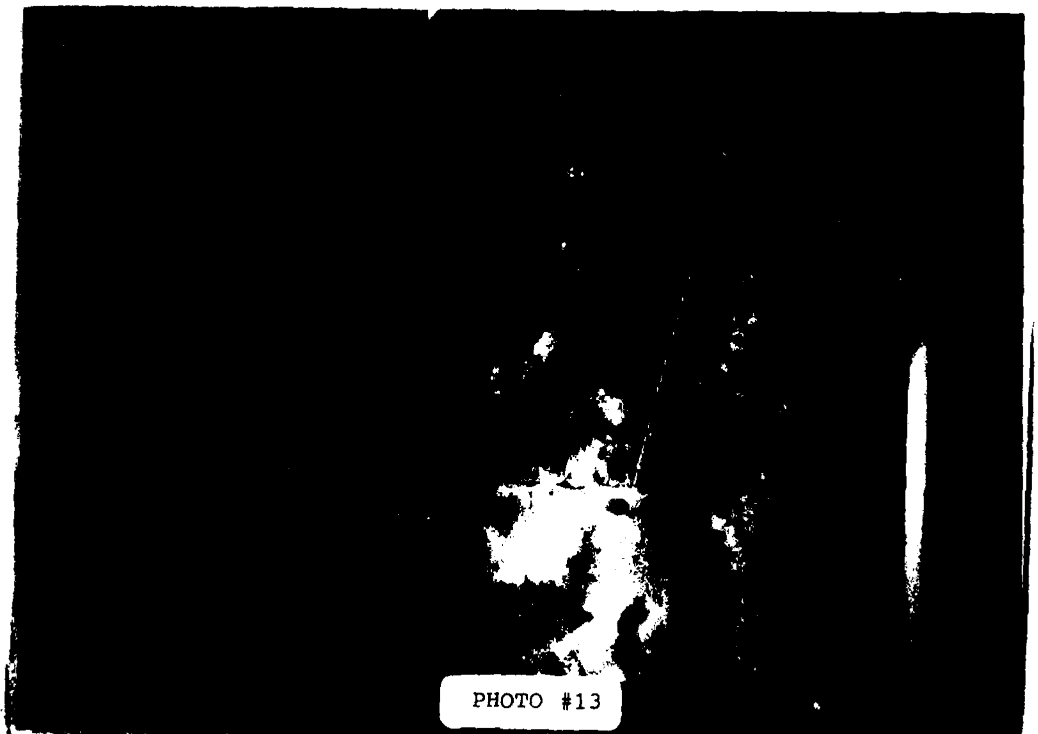


PHOTO #13

4.1.3 Structural Condition Assessment

Generally, Pier 2 is in good condition. Structural calculations indicate that the remaining pile capacity (73 tons, based on minimum metal thickness readings; see Appendix for Calculations), is still adequate to carry the imposed loads (60 tons). The welded splice connections all appear to be sound.

Pile B-B in Bent 28, however, has been damaged (see Section 4.1.2). At some time in the past, this pile appears to have been impacted. Corrosion was thus accelerated in this region, creating the large hole now observed and allowing the spalling of the concrete fill.

4.1.4 Recommendations

The hole in Pile B-B should be repaired with wire fabric and epoxy grout. The cost to repair this pile based on current area prices is $1200.00. Continued deterioration can be expected until repairs are performed. No other repairs are recommended. This pier should be inspected again in 5 - 10 years.

4.2 QUAYWALL APRON

4.2.1 Description

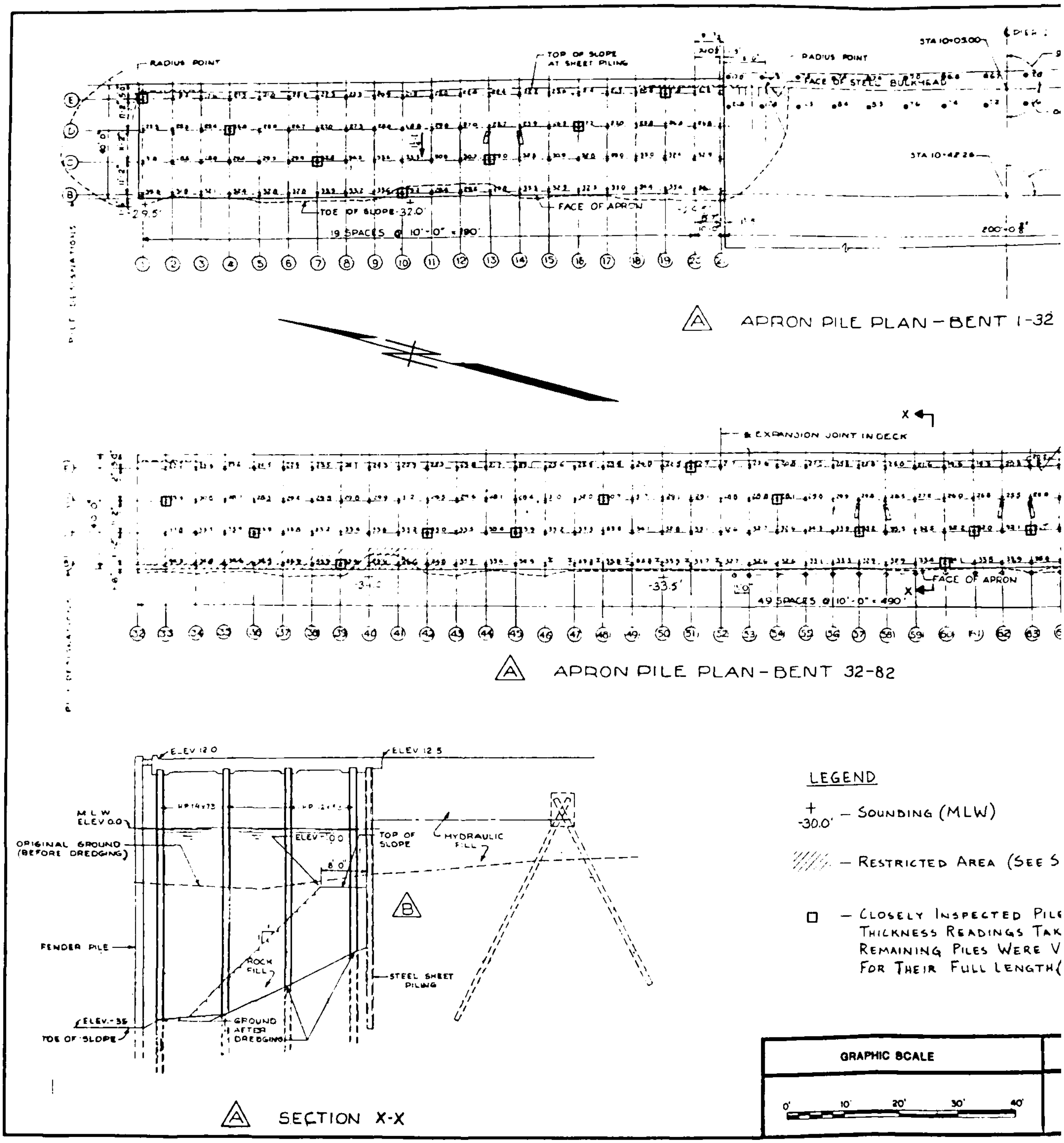
The quaywall apron is located on adjacent sides of the inshore end of Pier 2 in Coddington Cove. It provides vehicle access to and between Piers 1 and 2. It also provides for berthing of smaller Navy vessels.

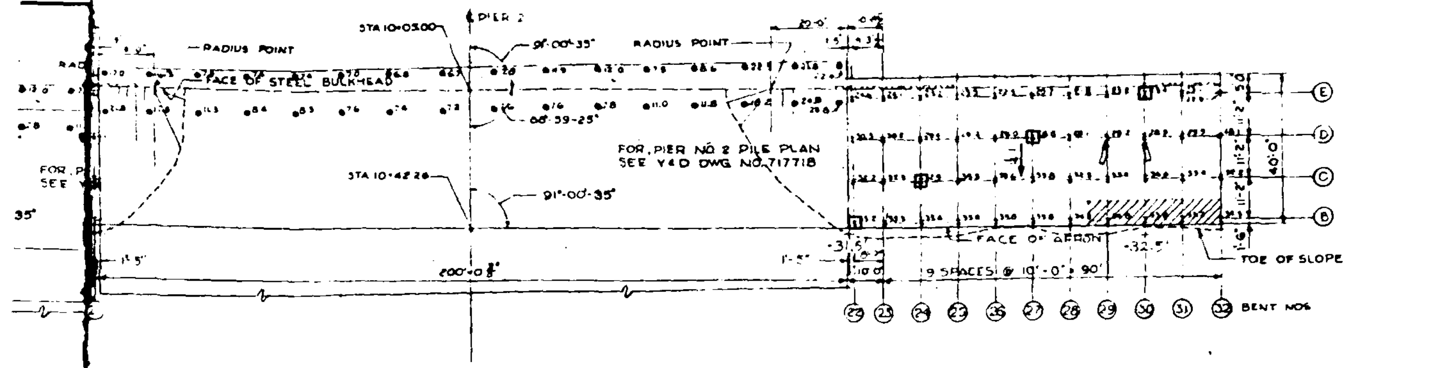
The apron, constructed around 1957, is 40' wide and consists of 21 bents (200' long) north of Pier 2 and 61 bents (595' long) south of Pier 2. A steel sheet pile bulkhead runs along the base of the apron. There are a total of 12 batter and 322 vertical bearing H-piles supporting the reinforced concrete decks. There are also 7 vertical replacement piles that were added in a later repair job. The perimeter piles and batter piles are HP 14 x 73 and the interior vertical piles are HP 12 x 53 (see Figure 7). In 1981, all the piles in the apron north of Pier 2 (Bents 1 to 21) were encased in concrete from the pile cap to elevation -5.0' below MLW. Zippered fabric forms were used to contain the concrete.

References:

Bureau of Yards and Docks
"Pier No. 2, Bulkhead and Utilities-Apron Pile Plan"
Y & D Dwg. No. 717733

Naval Facilities Engineering Command
Newport Education and Training Center
"Repairs to Bearing Piles-Concrete Apron North of Pier No. 2"
NAVFAC Dwg. No. 2044 444

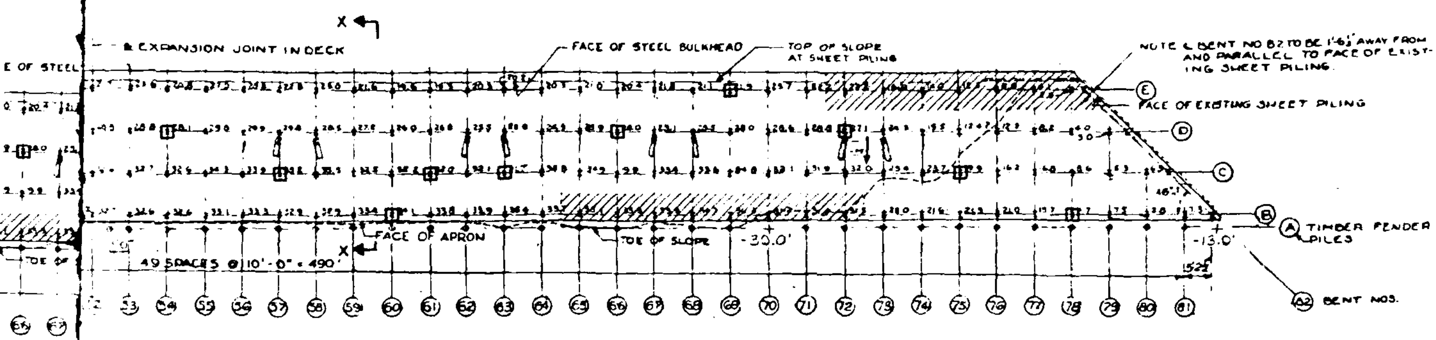

RADIUS POINT
TOP OF SLOPE AT SHEET PILING
RADIUS POINT
STA 10+05.00
FACE OF STEEL BULKHEAD
STA 10+42.28
29.5'
TOE OF SLOPE -32.0'
FACE OF APRON
19 SPACES @ 10'-0" = 190'
200'-0 1/8"
APRON PILE PLAN - BENT 1-32
EXPANSION JOINT IN DECK
-34.0
-33.5'
FACE OF APRON
49 SPACES @ 10'-0" = 490'
APRON PILE PLAN - BENT 32-82
ELEV 12.0
ELEV 12.5
M.L.W. ELEV 0.0
HP 14x73
ORIGINAL GROUND (BEFORE DREDGING)
ELEV 0.0
TOP OF SLOPE
HYDRAULIC FILL
8.0'
FENDER PILE
ROCK FILL
STEEL SHEET PILING
ELEV -35
TOE OF SLOPE
GROUND AFTER DREDGING
SECTION X-X
LEGEND
-30.0' – SOUNDING (MLW)
– RESTRICTED AREA (SEE S
– CLOSELY INSPECTED PIL
THICKNESS READINGS TAK
REMAINING PILES WERE V
FOR THEIR FULL LENGTH(
GRAPHIC SCALE
0' 10' 20' 30' 40'



APRON PILE PLAN – BENT 1-32

NOTE. PILES ON LINES C, D, E, E TO BE 12BP53
BATTER PILES AND PILES ON LINE B TO BE 14BP73

(A) NUMBERS ADJACENT TO PILE LOCATIONS ON APRON PILE PLAN SHOW THE DISTANCE BELOW MEAN LOW WATER TO GROUND (DREDGING COMPLETED) AS RECORDED ON PILE DRIVING RECORDS IN JUNE 1957 AND MAY BE APPROXIMATE ONLY



...LAN – BENT 32-82

LEGEND

+ -30.0' – SOUNDING (MLW)

//// – RESTRICTED AREA (SEE SECTION 4.2.4)

□ – CLOSELY INSPECTED PILE (I.E., METAL THICKNESS READINGS TAKEN). THE REMAINING PILES WERE VISUALLY INSPECTED FOR THEIR FULL LENGTH (SEE SECTION 3.2).

NOTE: TAKEN FROM BUREAU OF YARDS AND DOCKS "PIER NO. 2, BULKHEAD AND UTILITIES – APRON PILE PLAN" Y&D DWG. NO. 717733

GRAPHIC SCALE	GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.
0' 10' 20' 30' 40'	0' 20' 40' 60' 80'		NETC NEWPORT, RI — QUAYWALL APRON — FIG NO 7

4.2.2 Observed Inspection Condition

In the southern section of the quaywall apron, Bents 22 to 82, approximately 70% of the H-piles exhibited deflection of their flanges within 12 feet of the pile cap. The deflection varied from a slight waviness to a curling or a flattening of the flange against the web (see Photo #14). Some of this damage, particularly in the case of the perimeter piles, was due to impact (see Photo #15). In two areas along the face of the apron, from Bents 65 to 72 and from Bents 46 to 52, all the B piles were contorted, their pile caps were broken and spalled, and the concrete beam running between the piles was also broken and often spalled down to rebar (see Photo #16). Pile B of Bent 68 was broken away from its pile cap and displaced several feet toward Pile C. From Bents 46 to 52, all but one of the B piles were broken out of their pile caps (see Photo #17). New HP 14 x 73 piles had been added at some time in the past to compensate for this damage. Each replacement pile had been driven through the deck just north of the damaged pile's pile cap (except in Bent 46, where it is just south of the pile cap) and about 6" interior to the damaged pile (see Photos #18 and #19, and Figure 7). The hole in the concrete deck had been patched, and a steel plate was placed against the patch between the replacement pile head and the concrete deck (see Photo #20). Although these replacement piles were newer, they were in the same general condition as the interior piles of the apron.

All the piles in the southern section showed thinning of the flanges around MLW, and in many cases the flanges were partially gone. In the worst instances, the flanges necked down to the web around MLW (see Photo #21). Likewise, there was also a hole in the web at the same elevation (see Photo #22). A little over 10% of the piles in this section of apron had holes in the web around MLW (see Photos #23,#24 and #25).

Photo #14: Typical Deflection of Flange and Splash Zone Corrosion on an Interior Pile (Quaywall Apron).

Photo #15: Spalled and Broken Pile Cap and Impact Damaged Pile Head of Pile B in Bent 41 (Quaywall Apron). Note Welded Splice Connection in H-Pile Section.

Photo #16: Pile B in Bent 66, Showing Worse-Than-Usual Impact Deflection and Badly Spalled Concrete Beam (Quaywall Apron).

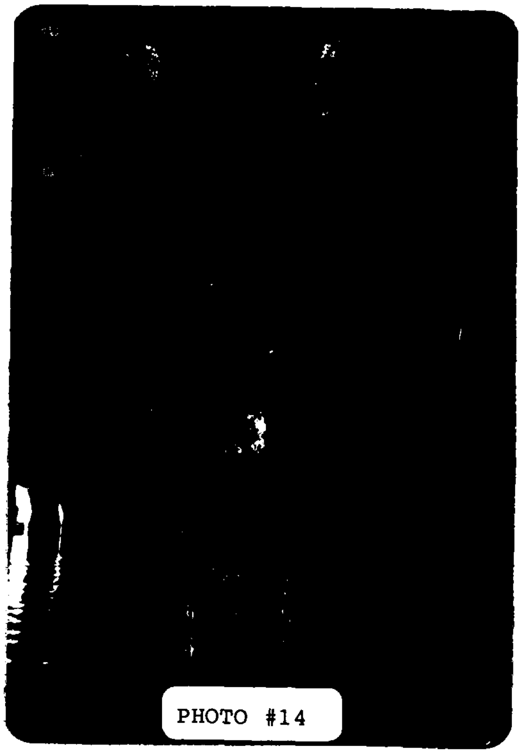


PHOTO #14


PHOTO #15


PHOTO #16

Photo #17: View from Bent 52 of the Quaywall Apron, Looking North Along the Perimeter Piles in the Area of the Worst Impact Damage.

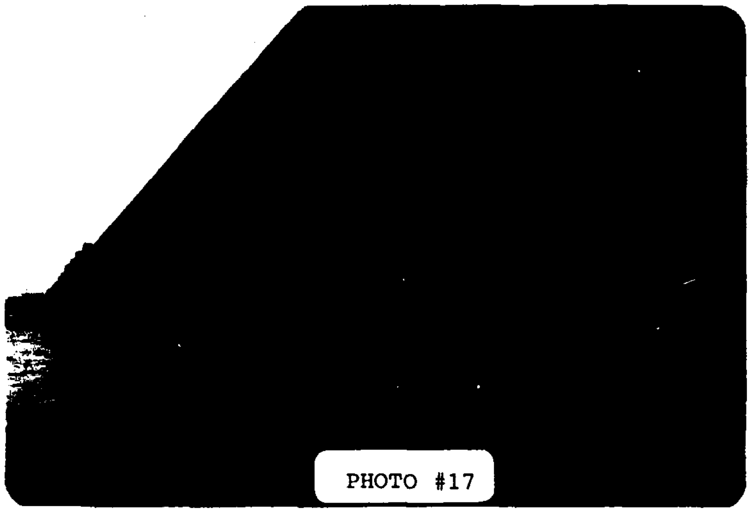


PHOTO #17

Photo #18: Imp'ct Damaged B Pile, Pile Cap an' Concrete Beam in Bent 52 of the Quaywall Apron. Note Replacement Pile in Back, Just North of the Pile Cap.

Photo #19: Impact Damaged and Displaced B Pile in Bent 50 of the Quaywall Apron. Note Replacement Pile in Back, Just North of the Pile Cap.

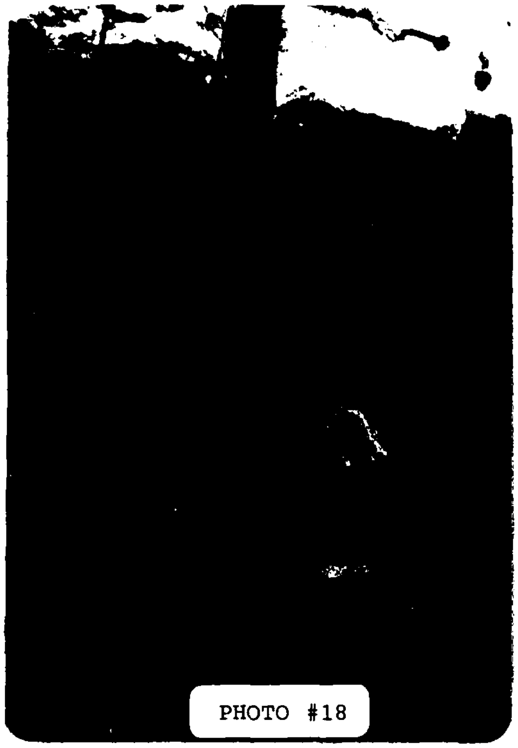


PHOTO #18


PHOTO #19

Photo #20: Typical View of Replacement Pile Head, Showing Patch in Concrete Deck and Steel Plate (Bent 50, Quaywall Apron).

Photo #21: Pile E in Bent 78 of the Quaywall Apron, Showing Flanges Corroded Down to the Web Around Mean Low Water.

Photo #22: Different View of Same Pile and Elevation in Photo #21, Showing Hole in Web.

PHOTO #20

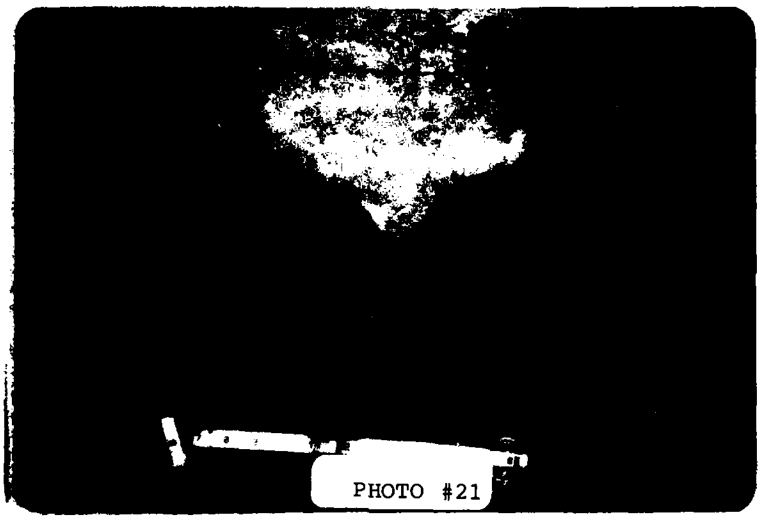


PHOTO #21

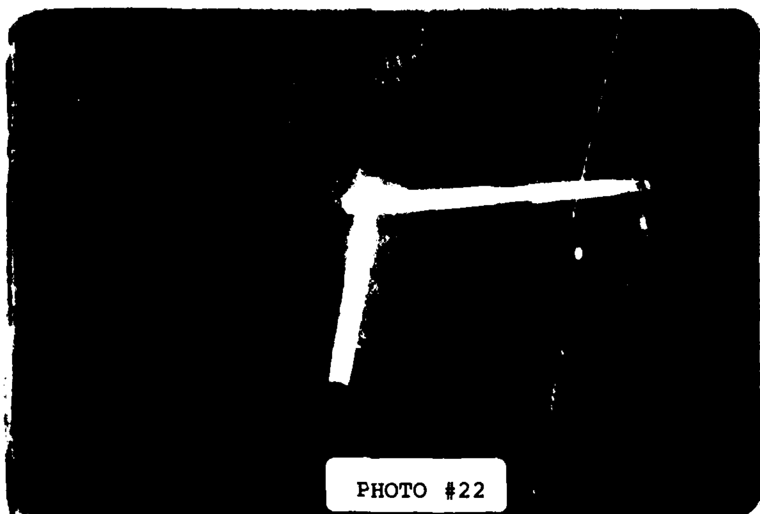


PHOTO #22

Photo #23: Pile D in Bent 78 of the Quaywall Apron, Showing a Hole in the Web Around Mean Low Water and a Slight Deflection of the Flanges.

Photo # 24: Pile E in Bent 76 of the Quaywall Apron, Showing a Hole in the Web and Corroded Flanges Around Mean Low Water.

Photo # 25: Pile B in Bent 48 of the Quaywall Apron, Showing a Hole in the Web and Corroded Flanges Around Mean Low Water.

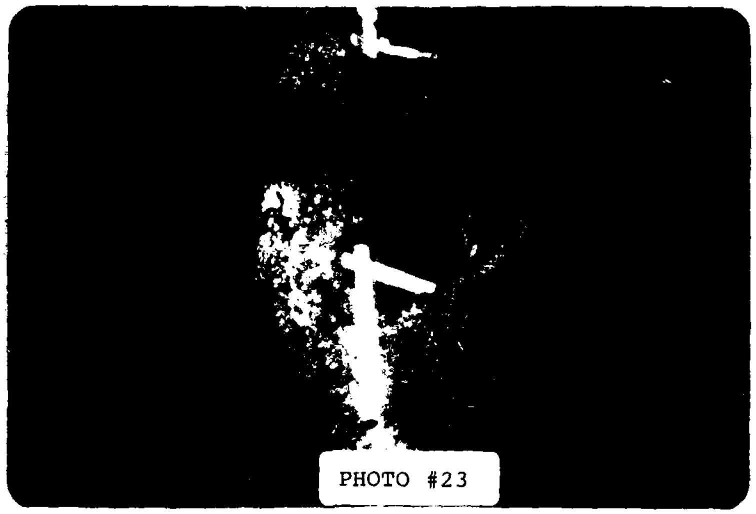


PHOTO #23

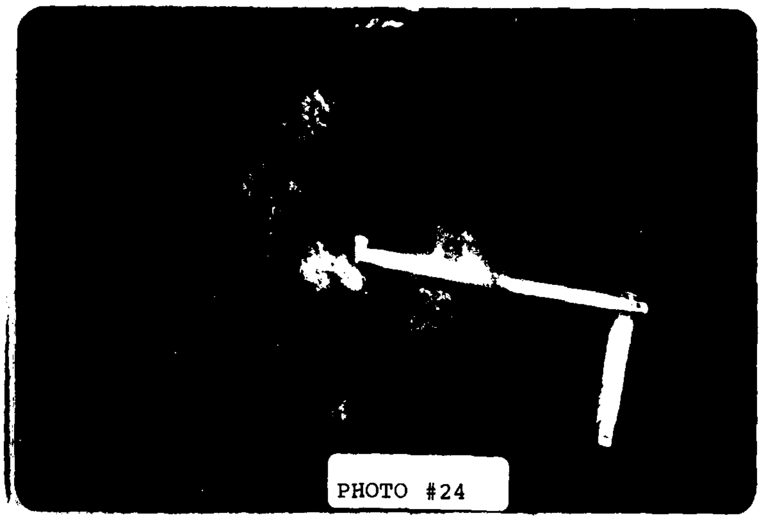


PHOTO #24

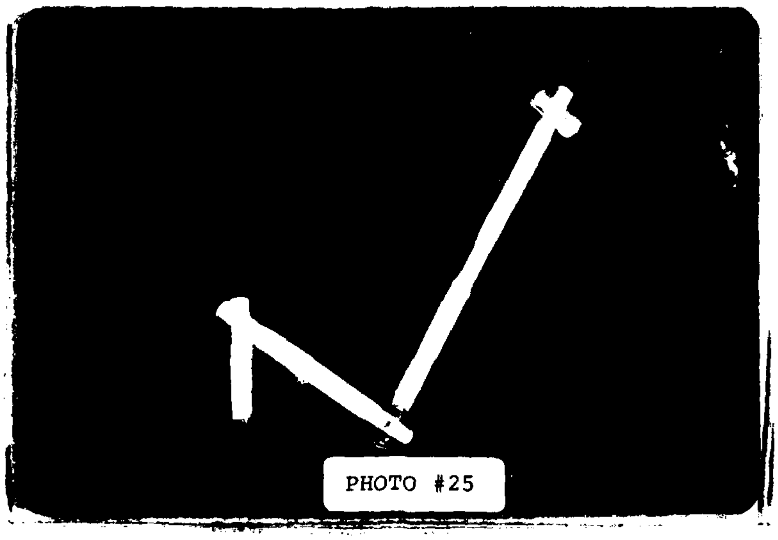


PHOTO #25

Many of the piles (probably over 50%) had welded splice connections within 5' - 10' of the pile cap. These connections were generally in adequate condition, with only two exceptions. On Pile B in Bent 75, the connection had been ripped for 2"- 3" into the flange (see Photo #26). The flange had been deflected at some time, and the rip was probably caused by that deflection. In the second exception, Pile B in Bent 64, the weld had deteriorated to the point that it could be knocked out with a hammer, and spaces could be seen between the two pile sections (see Photo #27).

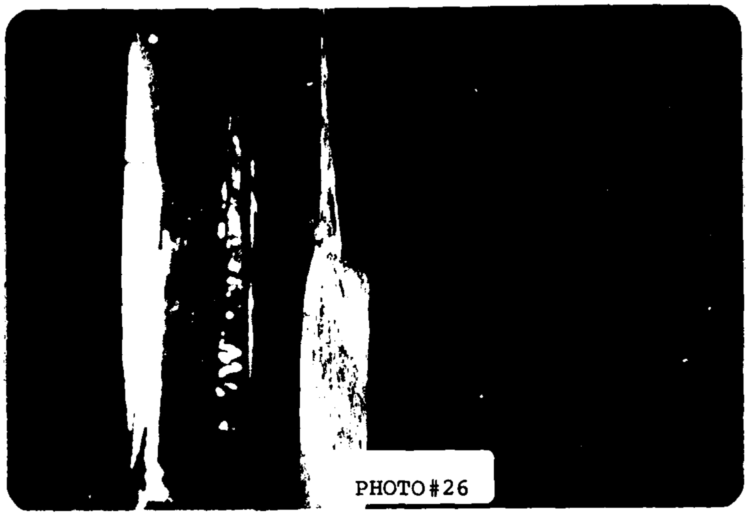
Steel thickness readings were taken on 18 piles in the region of severest corrosion (MLW to elevation +1.0') and on two piles from elevation +2.0' or +4.0' down to the mudline. In the area around MLW, the range of metal loss for the flanges was 23.2% to 100% with an average loss of 35% and for the webs was 9.4% to 100% with an average loss of 35% (see Appendix for actual readings).

Soundings taken along the perimeter of the southern section of apron ranged from -13.0' to -34.0' below MLW.

In the northern section of the quaywall apron, Bents 1 to 21, concrete jackets had been placed around the H-piles from the pile cap to around elevation -5.0' as a protection against further corrosion. The fabric bags that were used to form these jackets were all evident and in good condition, indicating the recency of the repair. Apparently, there was a tidal current running when some of the fabric forms were filled, since several of the jackets had pronounced S-curve shapes (see Photo #28). On one pile (Bent 19, Pile C), the form had ripped about a foot above the base of the jacket, causing a gap in the concrete approximately one foot wide around the circumference of the jacket, exposing the steel H-pile. Another problem encountered was in Bent 15, Pile D, where the form had ripped open for most of its length, exposing the steel reinforcing and standoffs (see Photo #29). Some

Photo #26: Ripped Welded Splice Connection Around El.+5.0' of the South Outboard Flange of Pile B in Bent 75 (Quaywall Apron).

Photo #27: Deteriorated Welded Splice Connection Around El.+3.0' of Pile B in Bent 64 (Quaywall Apron). Note Light Visible Through Welded Connection.



PHOTO#26

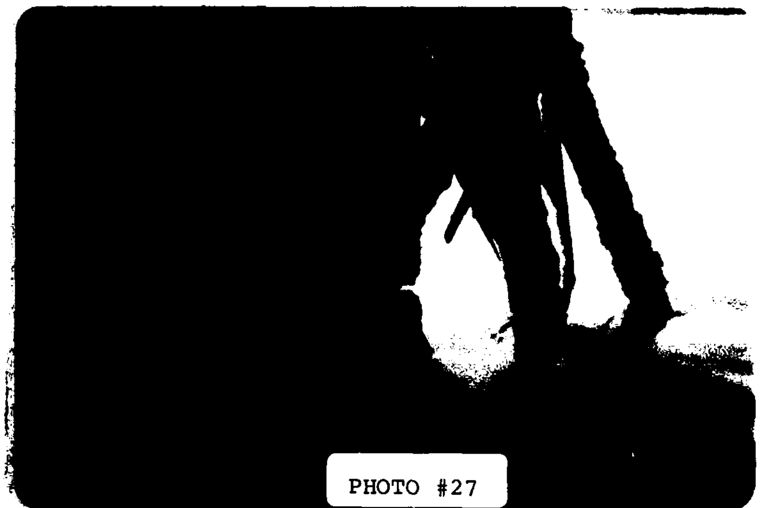


PHOTO #27

Photo #28: Pronounced S-Curve Shape of Concrete Jacket of Pile E in Bent 2 of the Quaywall Apron.

Photo #29: Ripped Fabric Form on Pile D of Bent 15 of the Quaywall Apron, Which Prevented the Complete Concrete Jacket from Being Placed.

PHOTO #28

PHOTO #29

concrete had been poured into the base of the form, but there was a 2"-3" gap around the circumference of the concrete abcut a foot above the base (see Photo #30).

The H-piles exposed below the jackets were in similar condition to those of the southern section of apron. At this elevation, the only condition of note was the expected corrosion of the steel. Steel thickness readings taken on 7 piles around elevation -6.0' indicated a range of metal loss on the flanges of 6.1% to 43.8% with an average loss of 18% and on the webs of 2.2% to 20.9% with an average loss of 15% (see Photo #31; also, see Appendix for actual readings).

Soundings taken along the perimeter of the northern section of apron ranged from -29.5' to -32.0' below MLW.

Photo #30: A 2"-3" Gap in Base of Concrete Jacket of Pile D in Bent 15, Around El.-4.0' (Quaywall Apron).

Photo #31: *View of Area* Cleaned *for* Steel Thickness Measurements Just Below the Concrete Jacket, Around El.-6.0' (Pile D, Bent 16 of Quaywall Apron).

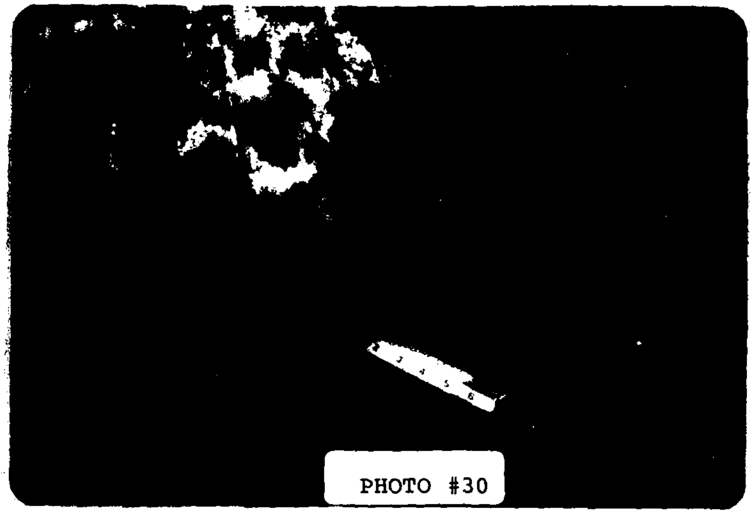


PHOTO #30

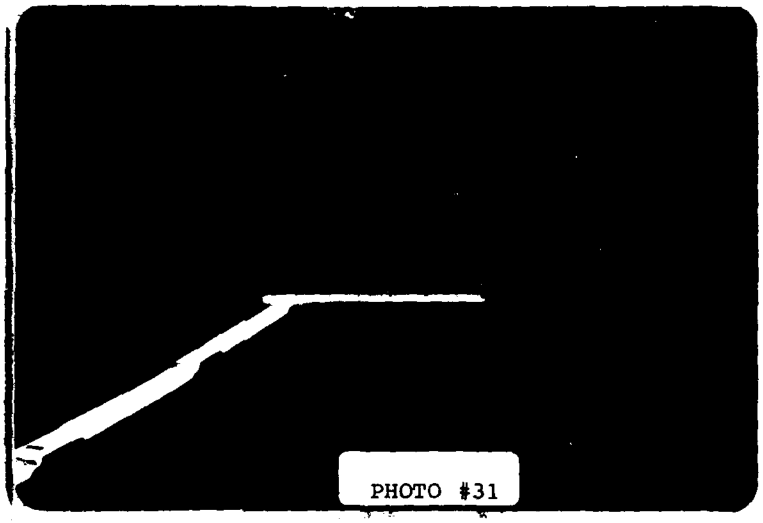


PHOTO #31

4.2.3 Structural Condition Assessment

The southern section of quaywall apron is in marginal condition. Based on the inspection, steel thickness measurements and structural analysis calculations, it is estimated that approximately 35% of the steel H-piles have corroded to the point that they no longer have enough residual capacity to carry the assumed original live load of 600 PSF (see Appendix for Calculations). In addition, approxim.tely 35% of the perimeter piles exhibit enough deflection and impact damage to warrant a similar judgement concerning their load-carrying capacity (this is excluding the 7 perimeter piles that were replaced with newer HP 14 x 73 piles).

The remainder of the piles in the southern section appear to be in adequate condition to handle the design live load (see Appendix for Structural Analysis Calculations). The welded splice connections are generally adequate (see Section 4.2.2 for the two exceptions), and the observed deflection of flanges is not severe enough to substantially affect the carrying capacity of the piles.

In the northern section of quaywall apron, corrosion of the exposed portion of H-pile below the jacket has not significantly reduced the carrying capacity of the piles. However, due to the recent jacketing of the piles, the area where maximum corrosion is expected has been covered by the concrete. This has prevented a comprehensive assessment of the residual capacity of the piles, and no conclusion can be made on their overall structural adequacy. The jackets themselves were in good condition, being fairly new. The one exception was Pile D on Bent 15, where the ripped fabric form prevented adequate placement of the concrete around the pile. Also, a few of the jackets had pronounced S-curve shapes which might have resulted in less than the minimum 4" coverage of concrete around the welded wire fabric and H-pile called for in the repair drawings (NAVFAC Dwg.No.2044 444). In the future, as the concrete spalls, the H-pile may be exposed sooner than anticipated, allowing further corrosion, possibly at an accelerated rate.

4.2.4 Recommendations

Because of the structural inadequacy of a significant percentage of piles in the southern section of quaywall apron, due to corrosion, impact damage or both, the loading of the apron should be reduced from the assumed original live load of 600 PSF to a maximum live load of 100 PSF, until repairs can be effected (see Appendix for Calculations). This may not create any real operational problems, however, if the apron serves merely as a roadway for light vehicular traffic (up to a uniform live loading of 100 PSF). All live loads should be limited to a very short duration and should be restricted from areas where supporting piles have been severely reduced in capacity. These areas include from Bents 72 to 82 in the vicinity of the E piles, and from Bents 65 to 72 and Bents 29 to 42 in the vicinity of the B piles, as shown in Figure 7.

Those piles in the southern section of apron which are now structurally inadequate, due either to severe impact damage or corrosion, should be restored to their original capacity by posting with new steel H-pile sections of the appropriate size or placement of a 30" diameter structural concrete jacket. The estimated cost of posting with the appropriate steel H-pile section from the pile cap to elevation -5.0' is $1620/HP 12 x 53 post, $1920/HP 14 x 73 post or a total cost of $176,000. The estimated cost of placing a structural concrete jacket from the pile cap to elevation -5.0' is $1985/pile or a total of $201,000.

The remaining piles in the southern section should be encased in non-structural concrete jackets from the pile cap to elevation -5.0' to protect them from further corrosion. Based on the placement of 28" diameter jackets, the estimated cost for this repair is $1320/pile or a total of $194,000.

In the northern section of the apron, the only repair recommended is the replacement of the ripped fabric form on Pile D of Bent 15 and placement of the concrete. The cost for this should already be covered in the original contract for the repair work.

TABLE OF CONTENTS FOR APPENDIX

FOOTNOTES

1. Naval Facilities Engineering Command, MASTER PLAN, Newport Education and Training Center, Newport, Rhode Island; Section VII, pp. 4-5.

2. *Ibid.*, Section VII, p.4.

3. *Ibid.*, Section II, p. 5.

4. *Ibid.*, Section III, p. 13.

5. *Ibid.*, Section III, p. 10.

REPAIR COST ESTIMATE

Pier 2

1) Patch hole in pile with epoxy compound and welded wire fabric.
MATERIAL COST =

$ 350.00 (100 in^2 hole)

Diver clean hole, install wire mesh, and place epoxy (1 day)
DIVER, TENDER, GEAR =

$ 840.00 / day

TOTAL COST FOR REPAIR $1,200.00

REPAIR COST ESTIMATE

Quaywall Apron

1) Restore H-piles to original capacity by replacing severely corroded sections with new steel pile posts from pile cap to elevation -5.0':

   a) Removal of corroded section of existing pile: $100/pile
   b) Pile cap and bottom connections: $500/pile
   c) Material, fabrication and installation:
      HP 12 x 53: $1.00/lb x 53lb/LF x 15LF (average length) =$795/pile
      HP 14 x 73: $1.00/lb x 73lb/LF x 15LF (average length) =$1095/pile
   d) Epoxy coating for new steel:
      $15/LF x 15LF = $225/pile
   e) Total Costs:
      Total Cost/HP 12 x 53 pile = $1620/pile
      Total Cost/HP 14 x 73 pile = $1920/pile
      Total Cost to post all structurally inadequate piles=
      $1620/pile x 62 piles + $1920/pile x 39 piles=
      $176,000

2) Restore H-piles to original capacity by placing a 30" diameter structural concrete jacket from the pile cap to elevation -5.0':

   a) Clean pile, add steel reinforcing and welded wire fabric, install form and place concrete:
      $735/cy concrete x .18 cy/LF x 15 LF/pile =
      $1985/pile
   b) Total Cost:
      $1985/pile x 101 piles = $201,000

3) Protect steel H-piles from further corrosion by placing

28" diameter concrete jackets from the pile cap to elevation -5.0':

a) Clean pile, add welded wire fabric, install form and place concrete:

   $550/cy concrete x .16 cy/LF x 15 LF/piles = $1320/pile

b) Total Cost:

   $1320/pile x 147 piles = $194,000

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JOB NETC NEWPORT ... ISLAND
SHEET NO 1 OF 2
CALCULATED BY DRC DATE 5/27/81
CHECKED BY DLP DATE 5/28
SCALE

COLUMN ANALYSIS - ASSUME K = .65 FIXED - FIXED
ℓ =

14" DIA PIPE PILE

Remaining Wall Thickness $\simeq$ .315

$$r = \frac{\sqrt{d_2^2 + d_1^2}}{4} = \frac{\sqrt{(14-(2\times.06))^2 + (14-(2\times.375))^2}}{4}$$

$$= 4.80 \text{ IN}$$

ASSUME ℓ = 35 DEPTH + 10' ABOVE MLW + 5' FIXITY = 50'

$$\frac{K\ell}{r} = \frac{.65\ (50\,ft)\ 12\,in/ft}{4.8\ in} = 81.25 \Rightarrow F_a = 15.24 \text{ KSI}$$

$$\text{X-SECT AREA} = -(14-(2\times.375))^2\frac{\pi}{4} + (14-(2\times.06))^2\frac{\pi}{4}$$

$$= 13.42 \text{ IN}^2$$

ALLOWABLE LOAD = 13.42 IN^2 × 15.24 KSI = $\underline{204.6^K = 102 \text{ TONS}}$

NOTES GREATER THAN DESIGN CAPACITY OF 60T (SEE Y&D DRAW. 717716)

— NO REDUCTION OF LOAD —

TRY 50 YRS AT SAME CORROSION RATE

r = 4.77 IN $\frac{K\ell}{r}$ = 81.66 F_a = 15.13

X-SECT AREA = 10.82 IN^2 ALLOW LOAD = $163^K = 81^T$

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WORST CASE THICKNESS = .230"

$$r = \sqrt{\frac{(14-(2\times.145))^2 + 14-(2\times.375)^2}{4}} = 4.77$$

$$r = 4.77 \qquad \frac{KL}{r} = \frac{.65\,(50')\,12''/'}{4.77} = 81.66$$

K_c = [illegible] 13 KSI

$$\text{AREA} = \text{[illegible]}\,4 - \text{[illegible]}\,.375))^2\,\frac{\pi}{4} + (14-(2\times.145))^2\,\frac{\pi}{4} = 9.74\ \text{IN}^2$$

ALLOWABLE LOAD = 9.74 IN^2 × 15.13 KSI = 147^K

$72^T > 60^T$

FORM 204 [illegible]

QUAYWALL APRON

COLUMN ANALYSIS - HP 14x73

Original Flange & Web Thickness = .506"

A.) Determination of Pile Capacity based on average web & flange thicknesses in the area of MLW (el. +2.0' to el. -2.0') - note, these are generally minimum thicknesses.

Avg. MLW web thickness = .280"
Avg. MLW flange thickness = .297"

Cross-sectional Area Remaining:

$$2(14.58'')(.297'') + (12.62'')(.280'') = 12.2\ in^2 \quad (\text{note: } A_{original} = 21.5\ in^2)$$

Radius of Gyration (r):

$r = \sqrt{I/A}$ $\quad A = 12.2\ in^2$
$I = 4I'$ $\quad I' = \Sigma A dx^2 + \Sigma \frac{bd^3}{12}$ (1/4 Section)
$= 28.78\ in^4 + 9.59\ in^4$
$= 38.37\ in^4$

$$\therefore r = \sqrt{4(38.37\ in^4)/12.2\ in^2} = 3.55\ in.$$

Allowable Stress (F_a), based on $\frac{Kl}{r}$:

K (fixed at both ends) = .65
$l = 49 ft. \times 12''/ft. = 588''$

$$\frac{Kl}{r} = \frac{.65 \times 588''}{3.55''} = 107.7 \cong 108$$

For $F_y = 36$ KSI, in AISC Table 1-36, $F_a = 11.94$ KSI.

$\therefore$ Present Capacity of Pile = 11.94 KSI x 12.2 in² = 145.7 K or <u>72.8 Tons</u>

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JOB 438-80 - NETC, Newport, RI - APRON
SHEET NO 2 OF 2
CALCULATED BY EWL DATE 10-15-81
CHECKED BY CLB DATE 10-27-81
SCALE

COLUMN ANALYSIS - HP 14 x 73 (cont'd)

B.) Allowable Loading of Piles.

1) Dead Load on average vertical pile:

Deck Area = 10'L x 7.1'W = 71 sf.

Deck Volume + Pile Cap Volume + Anchorage Cap =
(71 s.f. x 1'D) + (1.2'D x 2'W x 7.1'L) + (.2'D x 2'W x 3'L) =

Total Volume
89.24 c.f.

150 pcf x 89.24 c.f = 13,386 lb.

13,386 lb/71 s.f. = 188.5 psf => Dead Load/pile

2) Total Load

Assume a typical deck design live load of 600 psf

Total load = 600 psf + 188.5 psf ≅ 789 psf

3) Allowable loading

145,700 lbs (present pile capacity) x $\frac{\text{s.f.}}{789 \text{ lbs}}$ = 185 sf (Deck area pile can support)

Allowable area > Actual Area
185 sf > 71 s.f.

∴ Piles are structurally adequate.

FORM 204-1 Available from NEBS, Inc. Groton, Mass. 01450

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QUAYWALL APRON

COLUMN ANALYSIS - HP 12X53

Original Flange + Web Thickness = .436"

A.) Determination of Pile Capacity based on average web + flange thicknesses in the area of MLW (el. +2.0' to el. -2.0') - note, these are generally minimum thicknesses.

Avg. MLW web thickness = .252"
Avg. MLW flange thickness = .235"

Cross-sectional area remaining:

$2(12.04")(.235") + (10.90")(.252") = 8.4\ in^2$ (note: $A_{original} = 15.6 in^2$)

Radius of gyration (r):

$r = \sqrt{I/A}$ $A = 8.4\ in^2$ $I = 4I'$ $I' = \Sigma Ad_x^2 + \Sigma \frac{bd^3}{12}$
$= 12.77\ in^4 + 4.27\ in^2$
$= 17.04\ in^4$ (1/4 Section)

$\therefore r = \sqrt{4(17.04\ in.^4)/8.4\ in^2} = 2.85\ in.$

Allowable Stress (Fa), based on $\frac{Kl}{r}$:

K (fixed at both ends) = .65
$l = 39\ ft. \times 12"/ft. = 468"$

$\frac{Kl}{r} = \frac{.65 \times 468"}{2.85"} = 106.7 \cong 107$

For $F_y = 36$ KSI, in AISC Table 1-36, $F_a = 12.07$ KSI

$\therefore$ Present Capacity of Pile = 12.07 KSI × 8.4 in² = 101.4 K or 50.7 Tons

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COLUMN ANALYSIS - HP 12x53 (cont'd)

B.) Allowable Loading of Piles.

1) Dead Load on average vertical pile:

Deck Area = 10'L x 11.2'W = 112 sf

Deck Volume + Pile Cap Volume + Anchorage Cap =
(112 sf x 1'D) + (1.2'D x 2'W x 11.2'L) + (.2'D x 2'W x 3'L) =

Total Volume
140.08 cf.

150 pcf x 140.08 cf = 21,012 lb.

21,012 lb / 112 sf = 187.6 psf ⇒ Dead Load/pile

2) Total Load

Assume a typical deck design live load of 600 psf

Total load = 600 psf + 187.6 psf ≅ 788 psf

3) Allowable loading

101,400 lb (present pile capacity) x $\frac{\text{s.f.}}{788\text{ lb}}$ = 129 sf (Deck area pile can support)

Allowable area > Actual area
129 sf > 112 sf.

∴ Piles are structurally adequate.

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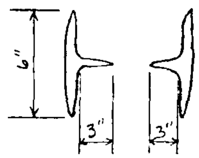
JOB 438-80 - NETC, Newport, RI - APRON
SHEET NO 3 OF 3
CALCULATED BY LWL DATE 10-15-81
CHECKED BY CLB DATE 10/27/81
SCALE

Column Analysis - HP 12X53 (cont'd)

C.) Determination of Pile Capacity based on minimum web + flange thicknesses and Allowable Loading.

1) minimum web thickness = 0"
minimum flange thickness = 0"



Avg. remaining web thickness = .205"
Avg. remaining flange thickness = .125"

Cross Section through
Area of Minimum Thickness

Cross-sectional Area Remaining:

$$2(6")(.125") + 2(3")(.205") = 2.73 \text{ in}^2$$

∴ Present Capacity of Pile = 12.07 KSI × 2.73 in² = 33.0 K or 16.5 Tons

2) Allowable loading:

$$33{,}000 \text{ lb.} \times \frac{\text{s.f.}}{788 \text{ lb}} = 41.9 \text{ s.f.}$$

Allowable area < Actual Area
41.9 s.f. < 112 s.f.

∴ Piles are not structurally adequate.

3) Allowable Deck Live Load:

33,000 lb − 21,012 lb (dead load) = 11,988 lb.

11,988 lb / 112 s.f. = 107.0 p.s.f. ≅ 100 psf ⇒ Allowable Live Load

NETC NEWPORT RHODE ISLAN[D]

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JOB NETC NEWPORT RHODE ISLAN[D]
SHEET NO 1 OF 2
CALCULATED BY DRC DATE 5/7/81
CHECKED BY ______ DATE ______
SCALE ______

PIPE PILE

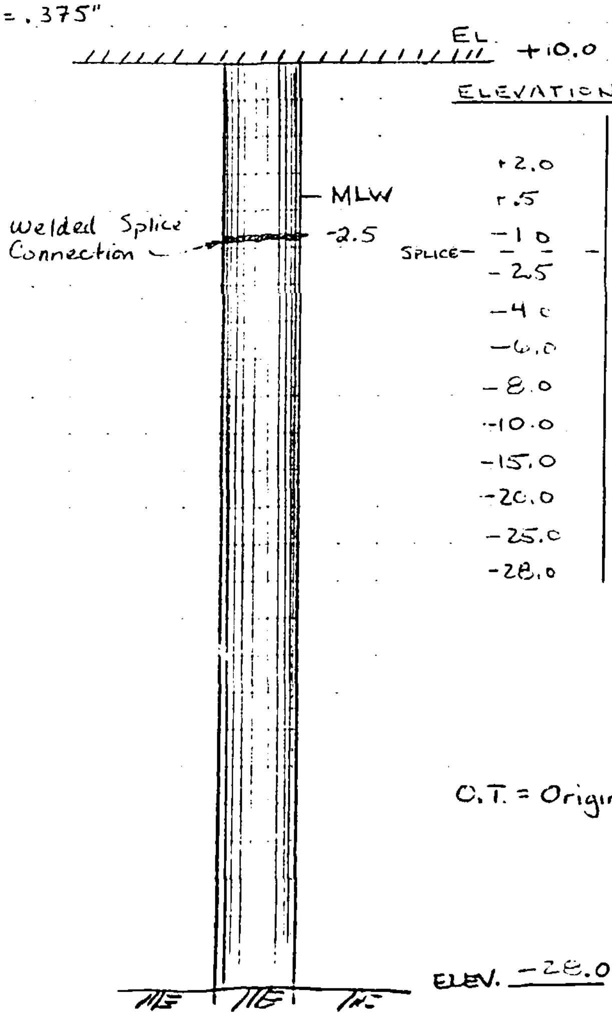
STEEL THICKNESS MEASUREMENTS

PIER NO. 2 **BENT NO.** 113 **PILE** D-D

PILE SIZE 14" DIA

ORIGINAL THICKNESS:
Above Splice = .500"
Below Splice = .375"

DATUM - MLW



ELEVATION (ft)	THICKNESS (in) N	E	S	W
+2.0	.255	.345	.435	.325
+.5	.435	.455	.445	.455
-1.0	O.T.	.375	.335	.315
SPLICE				
-2.5	.360	.140	.335	[illegible]
-4.0	.350	.330	.335	.340
-6.0	.340	.195	.320	.140
-8.0	.325	O.T.	.330	.310
-10.0	.310	.320	.330	.215
-15.0	.310	.300	.335	.360
-20.0	.315	.310	.315	.350
-25.0	.305	.330	O.T.	.315
-28.0	.310	.305	.320	.320

O.T. = Original Thickness

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JOB NETC NEWPORT RHODE ISLANI

SHEET NO 2 OF 2

CALCULATED BY DRC DATE [illegible]

CHECKED BY ______ DATE ______

SCALE ______

PIPE PILE

STEEL THICKNESS MEASUREMENTS

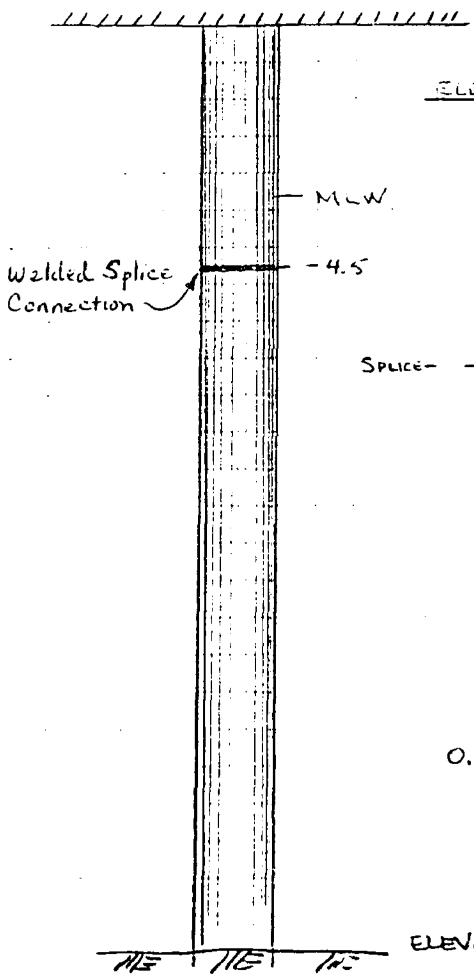
PIER NO. 2 BENT NO. 38 PILE D

PILE SIZE 14" DIA.

ORIGINAL THICKNESS:
Above Splice = .500"
Below Splice = .375"

DATUM - MLW

EL. +10.0



ELEVATION (FT)	THICKNESS (IN.) N	THICKNESS (IN.) W
+1.0	.305	.360
0.0	.245	.230
-1.0	.360	290
-2.0	.445	.470
-4.0	.230	.485
SPLICE-		
-6.0	O.T.	O.T.
-8.0	.345	.350
-10.0	.355	.340
-15.0	.345	.360
-20.0	.320	O.T.
-25.0	.335	.340
-30.0	.350	.325

O.T. = Original Thickness

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JOB 438-80 NETC, NEWPORT, RI

SHEET NO 1 OF 14

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

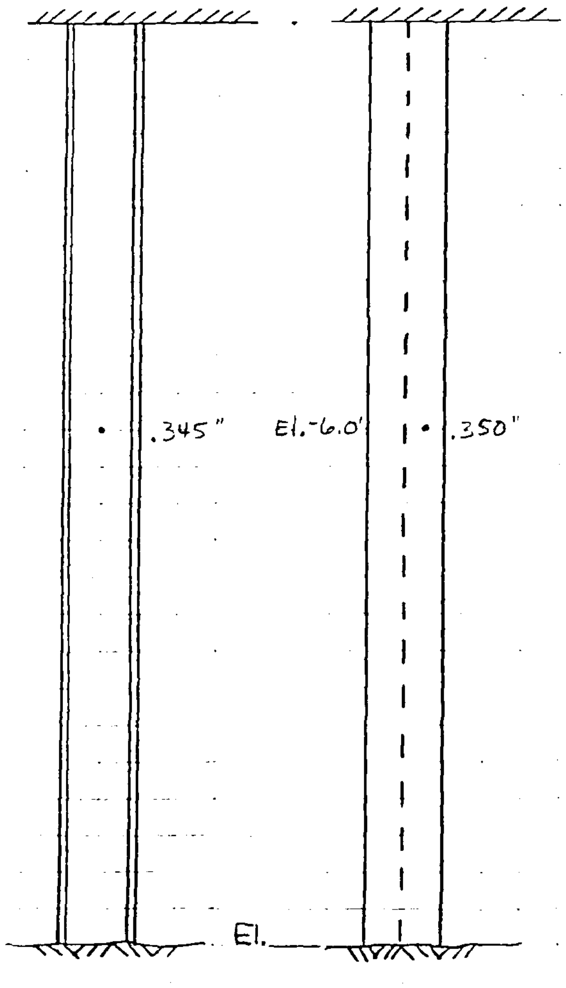
LOCATION Quaywall Apron

BENT 1 PILE E

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE

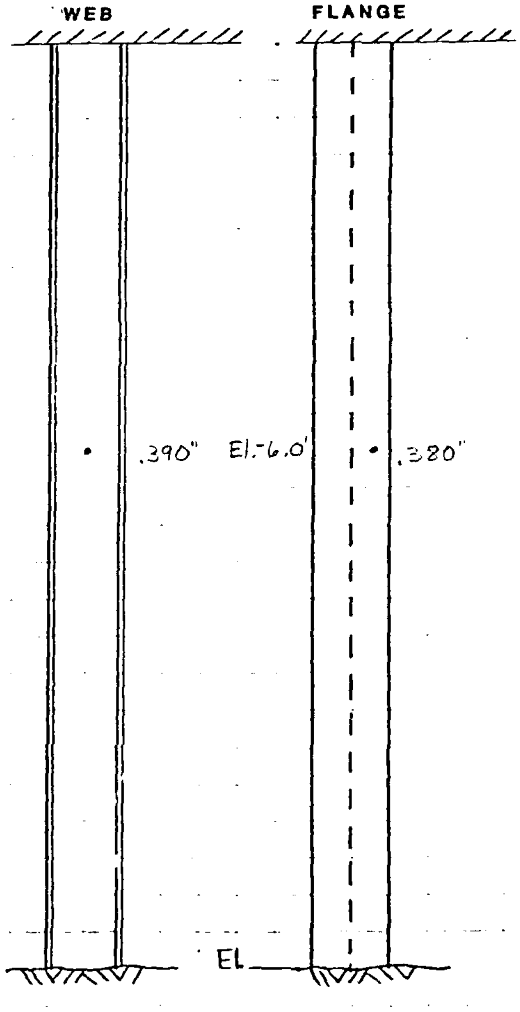


BENT 4 PILE D

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE



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SHEET NO 2 OF 14

CALCULATED BY ____ DATE ____

CHECKED BY ____ DATE ____

SCALE ____

H-PILE

STEEL THICKNESS MEASUREMENTS

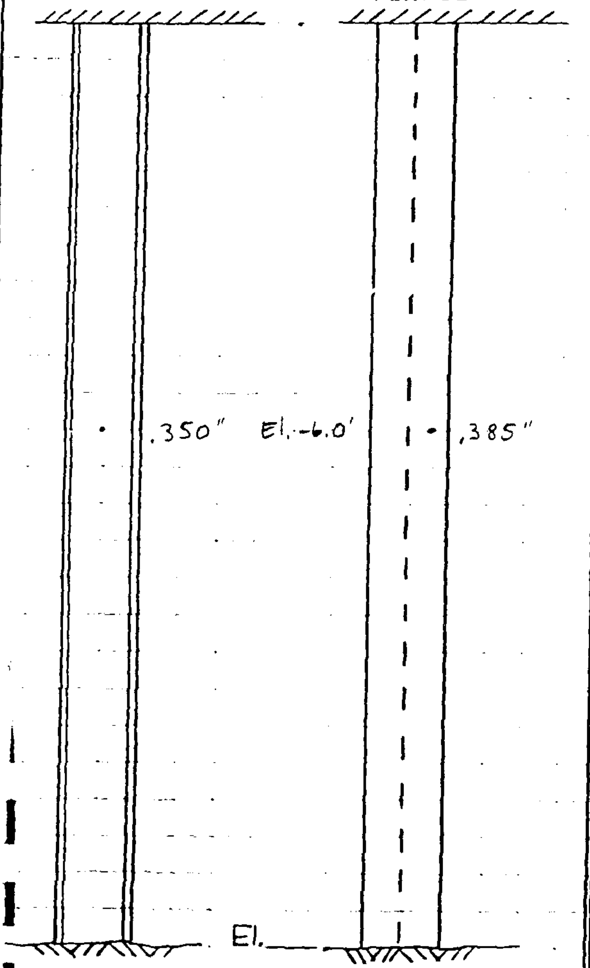
LOCATION Quaywall Apron

BENT 7 PILE C

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436"
WEB

ORIGINAL THICKNESS: .436"
FLANGE

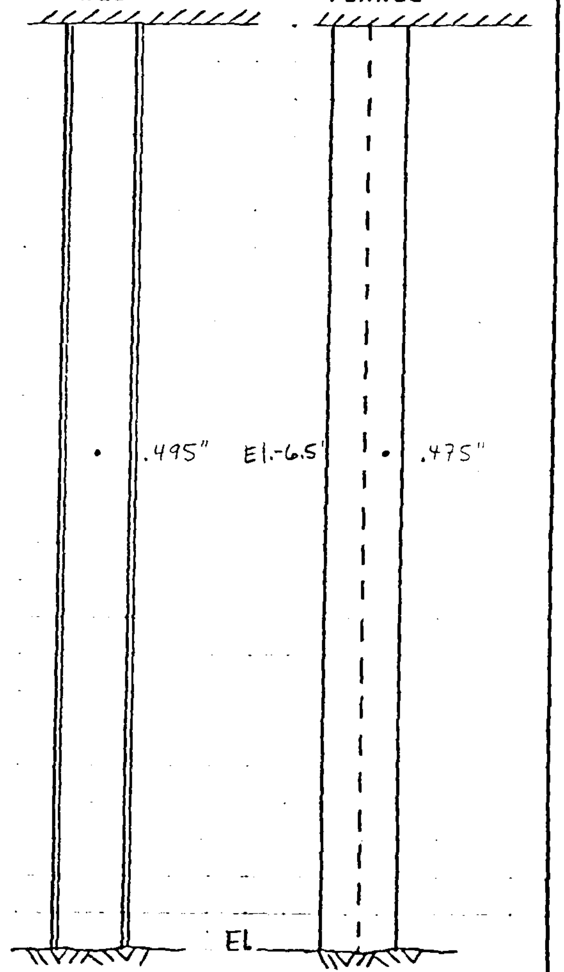


BENT 10 PILE B

PILE TYPE HP 14x73

ORIGINAL THICKNESS: .506"
WEB

ORIGINAL THICKNESS: .506"
FLANGE



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SHEET NO 3 OF 14

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

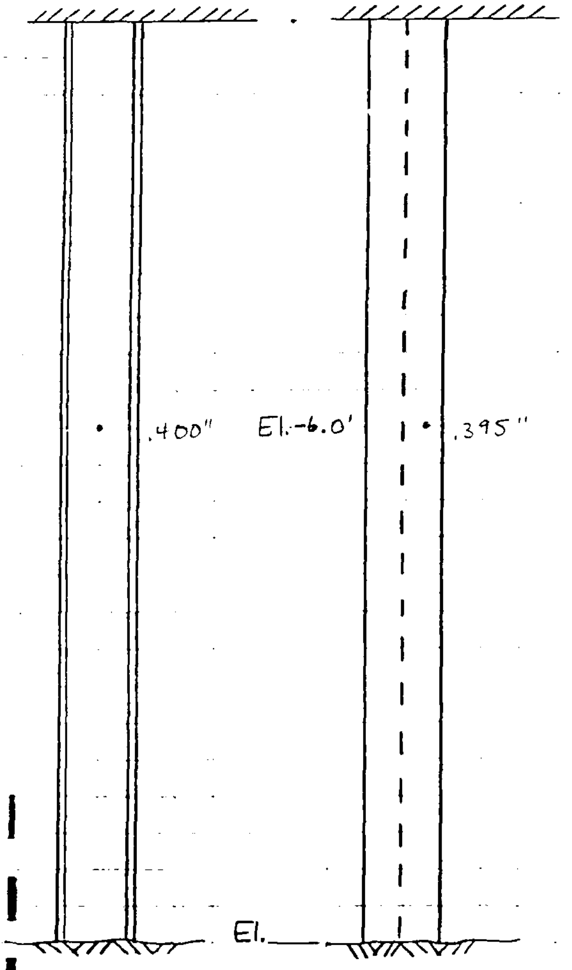
LOCATION Quaywall Apron

BENT 13 PILE C

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE

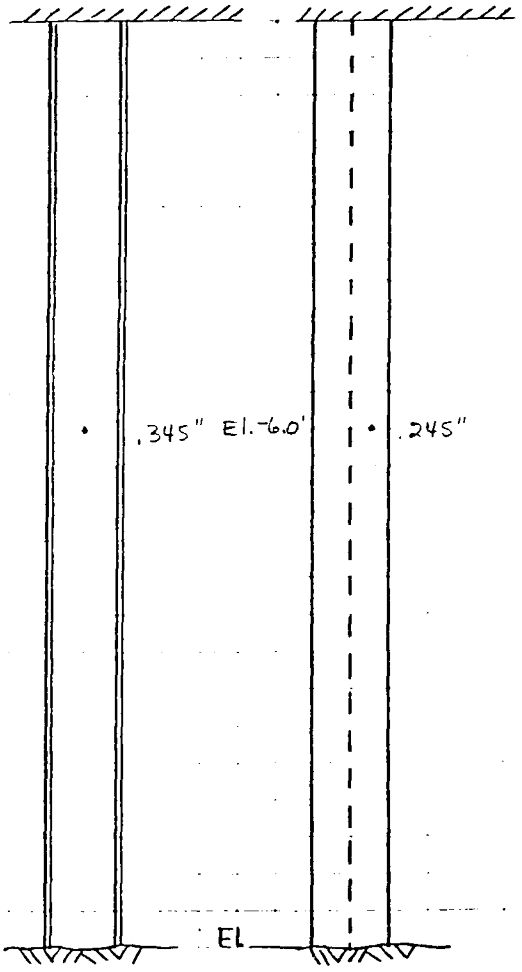


BENT 16 PILE D

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE



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SHEET NO 4 OF 14

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

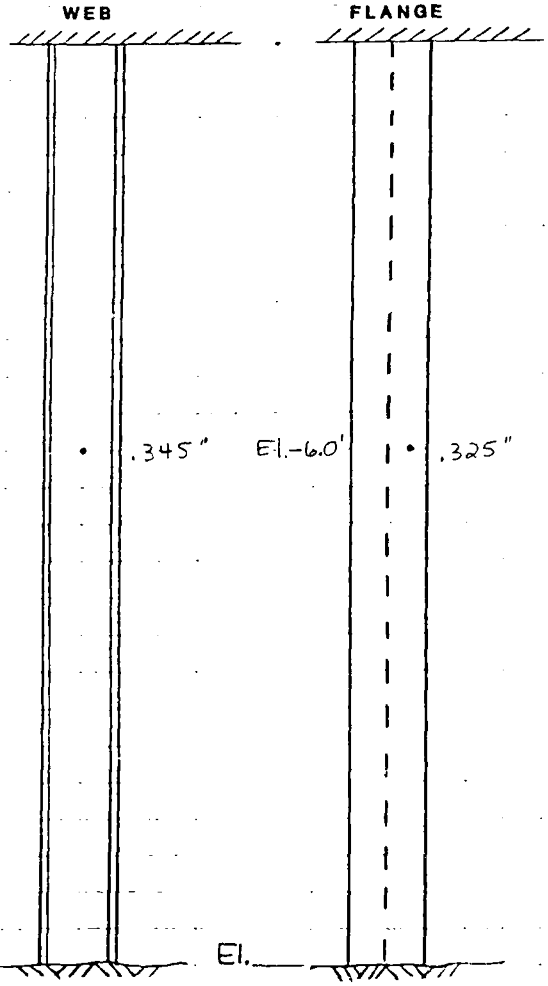
LOCATION QUAYWALL APRON

BENT 19 PILE E

PILE TYPE HP 12X53

ORIGINAL THICKNESS: .436"
WEB

ORIGINAL THICKNESS: .436"
FLANGE

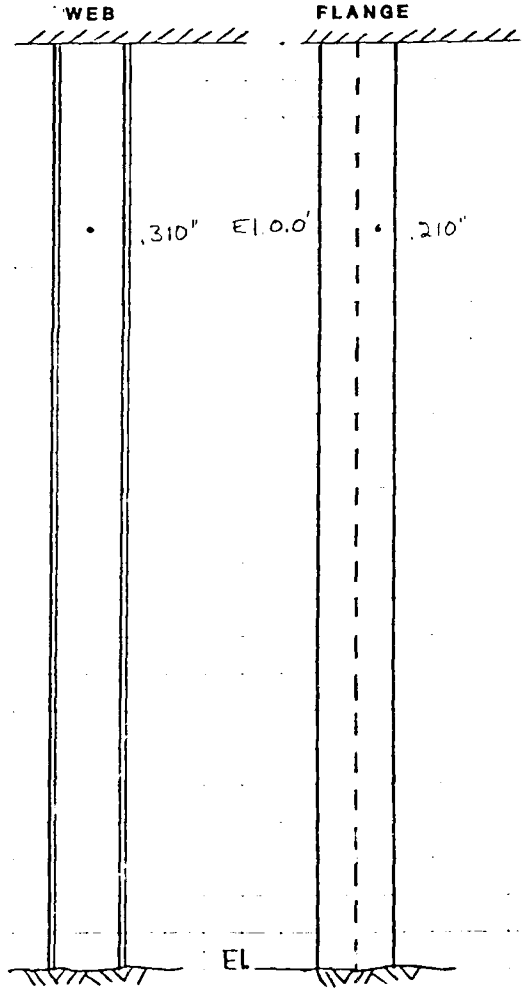


BENT 22 PILE B

PILE TYPE HP 14X73

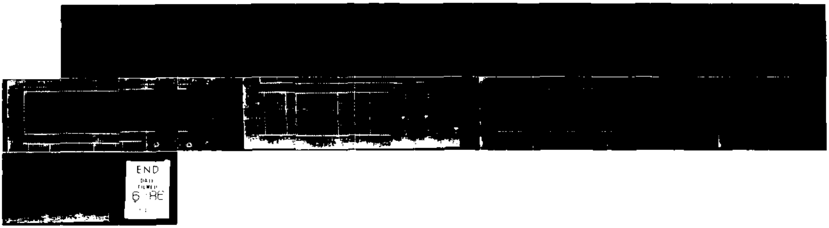
ORIGINAL THICKNESS: .506"
WEB

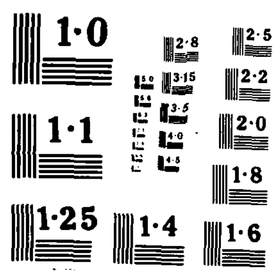
ORIGINAL THICKNESS: .506"
FLANGE



AD-A167 699 UNDERWATER FACILITIES INSPECTIONS & ASSESSMENTS AT NAVAL EDUCATION AND TR.. (U) CHILDS ENGINEERING CORP MEDFIELD MA MAY 81 CHES/NAVFAC-FPO-1-81 (10) 2/2

UNCLASSIFIED N62477-80-C-0102 F/G 13/2 NL




1·0
2·8
2·5
3·15
2·2
3·5
2·0
1·1
4·0
4·5
1·8
1·25
1·4
1·6

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JOB 438-80 NETC, NEWPORT, RI
SHEET NO 5 OF 14
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

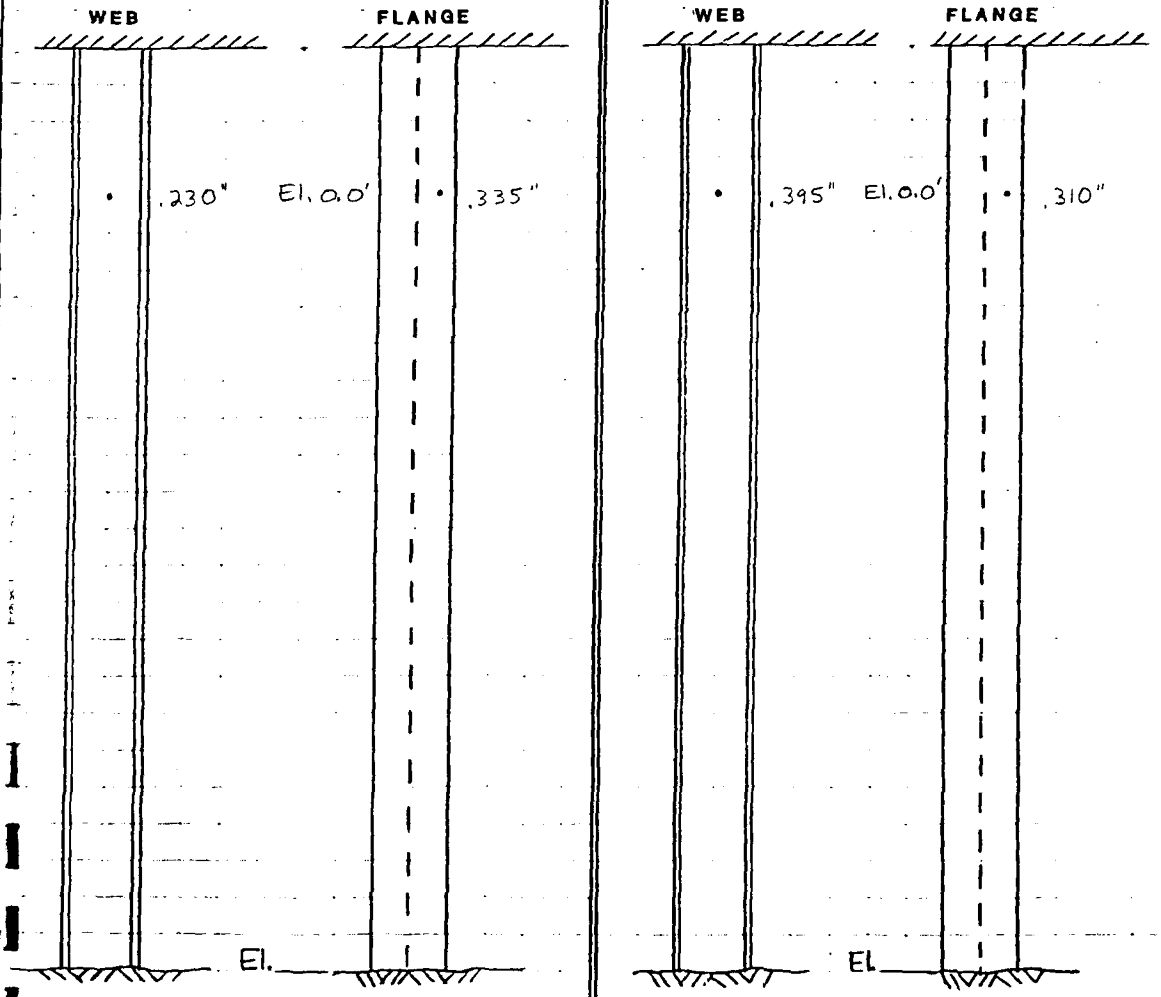
H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

BENT 24 PILE C
PILE TYPE HP 12x53
ORIGINAL THICKNESS: .436" (WEB)
ORIGINAL THICKNESS: .436" (FLANGE)

BENT 27 PILE D
PILE TYPE HP 12x53
ORIGINAL THICKNESS: .436" (WEB)
ORIGINAL THICKNESS: .436" (FLANGE)



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JOB 438-80 NETC, NEWPORT, RI

SHEET NO 6 OF 14

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

BENT 30 PILE E

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" (WEB)

ORIGINAL THICKNESS: .436" (FLANGE)

WEB		FLANGE
.340"	El. 0.0'	.295"

El. ______

BENT 33 PILE D

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" (WEB)

ORIGINAL THICKNESS: .436" (FLANGE)

WEB		FLANGE
.395"	El. +2.0'	7/16"
.305"	0.0'	3/16"
.400"	-1.5'	5/16
.395"	-3.0'	5/16"
.310"	-5.0'	5/16"
.320"	-6.5'	5/16"
.255"	-8.0'	3/8"
.280"	El -13.5'	3/16"

CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

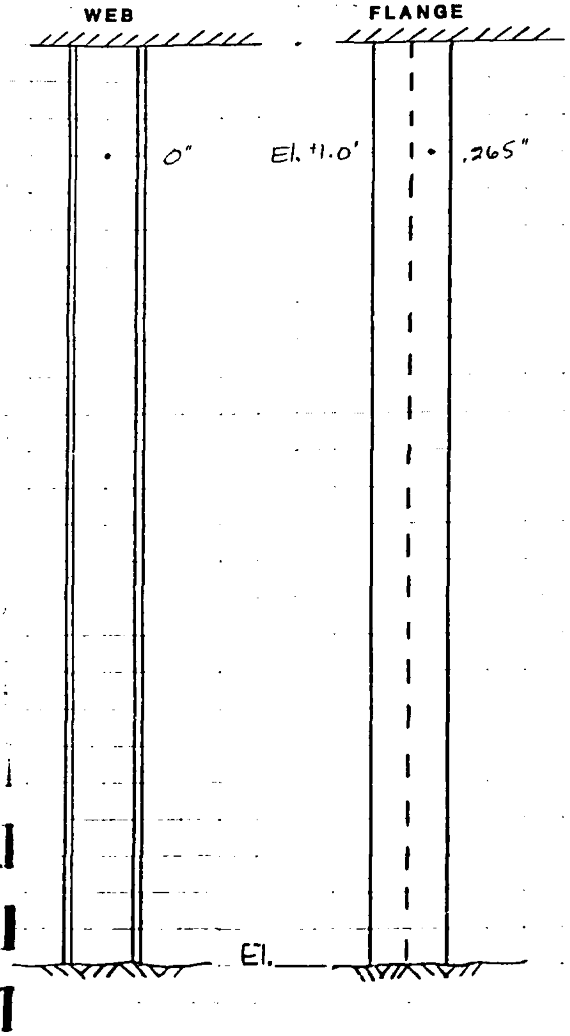
H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION: QUAYWALL APRON

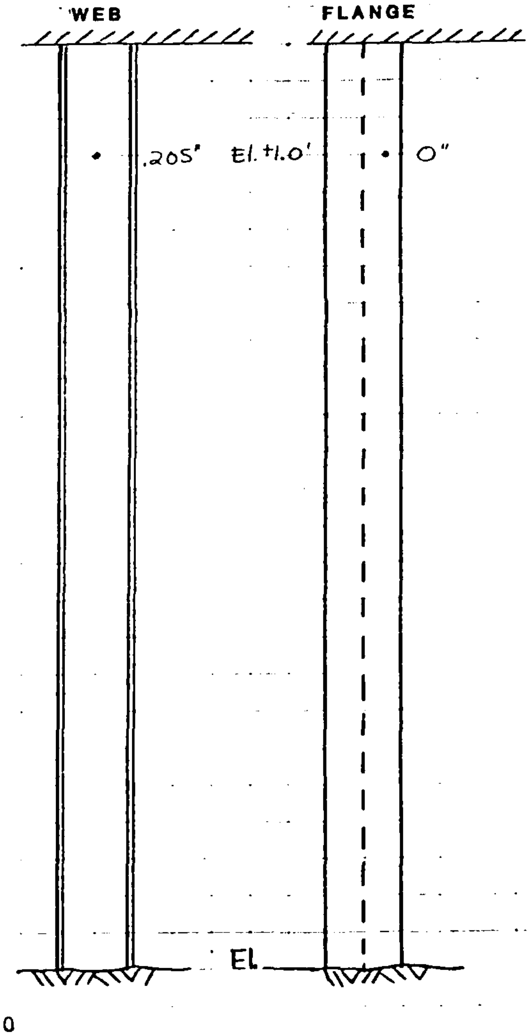
BENT 36 PILE C
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" (WEB)
ORIGINAL THICKNESS: .436" (FLANGE)



BENT 39 PILE B
PILE TYPE HP 14x73

ORIGINAL THICKNESS: .506" (WEB)
ORIGINAL THICKNESS: .506" (FLANGE)



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB 438-80 NETC, NEWPORT, RI

SHEET NO 8 OF 14

CALCULATED BY ____ DATE ____

CHECKED BY ____ DATE ____

SCALE ____

H-PILE

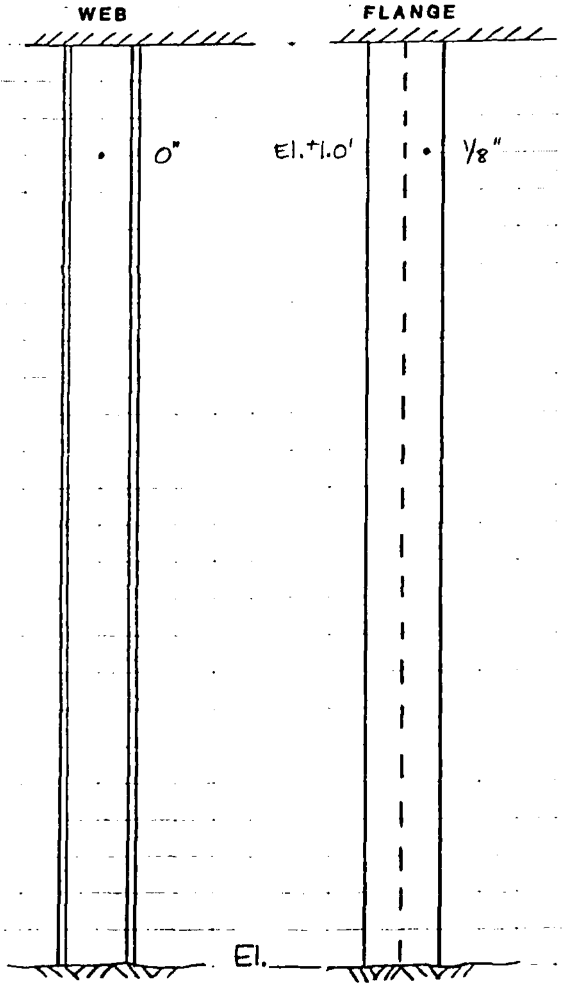
STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

BENT 42 PILE C
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

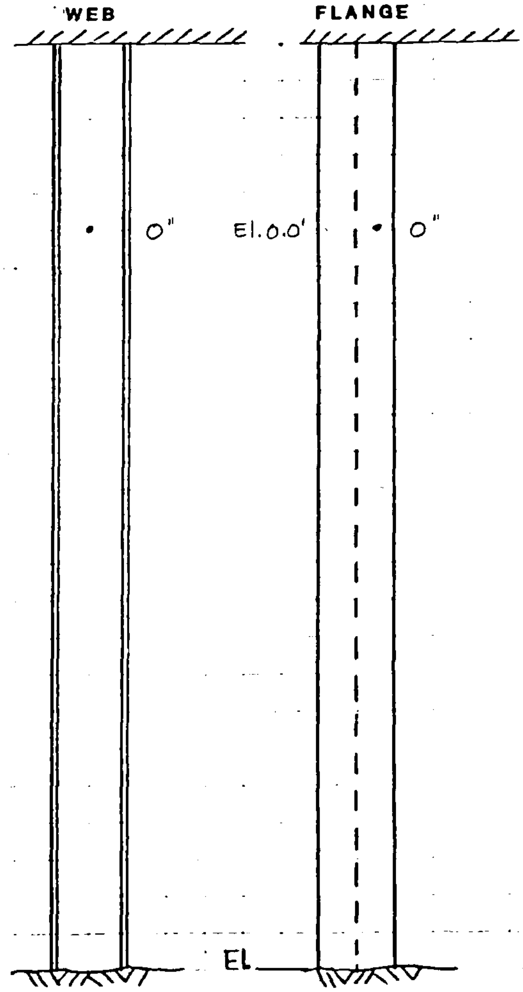
ORIGINAL THICKNESS: .436" FLANGE



BENT 45 PILE C
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

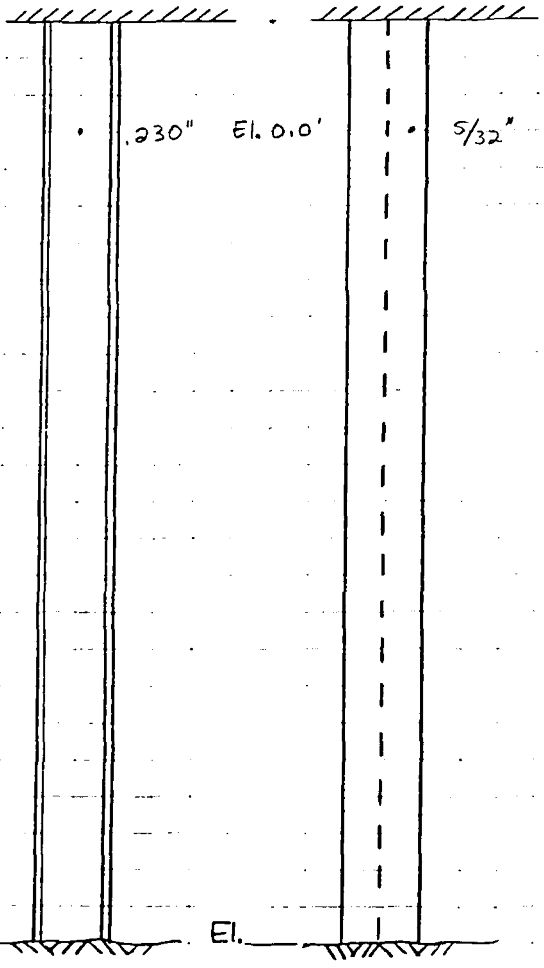
H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

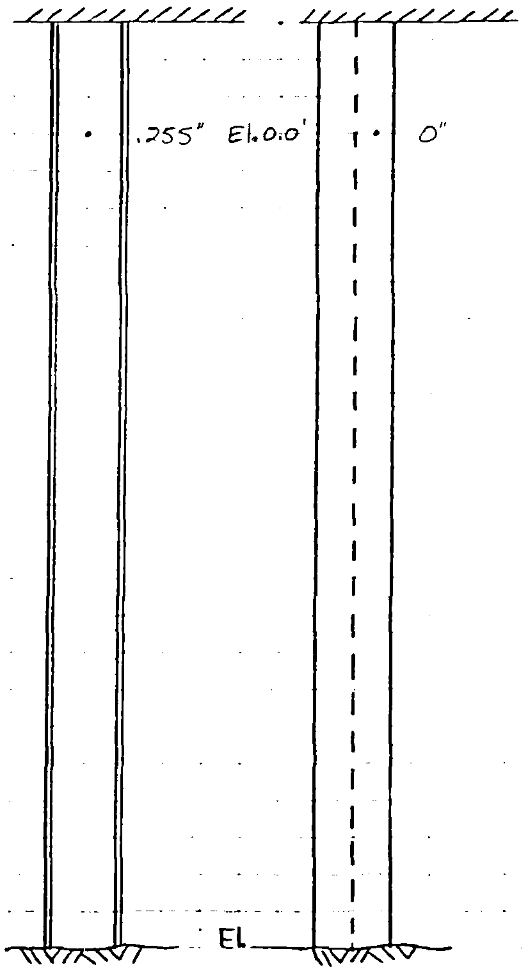
BENT 48 PILE D
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB
ORIGINAL THICKNESS: .436" FLANGE



BENT 51 PILE E
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB
ORIGINAL THICKNESS: .436" FLANGE



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB 438-80 NETC, NEWPORT, RI
SHEET NO 10 OF 14
CALCULATED BY ______ DATE ______
CHECKED BY ______ DATE ______
SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

BENT 54 PILE D
PILE TYPE HP 12x53

WEB ORIGINAL THICKNESS: .436"	Elevation	FLANGE ORIGINAL THICKNESS: .436"
.330"	El. 0.0'	1/4"
	El. ______	

BENT 57 PILE C
PILE TYPE HP 12x53

WEB ORIGINAL THICKNESS: .436"	Elevation	FLANGE ORIGINAL THICKNESS: .436"
.330"	El. +4.0'	.385"
.335"	+2.0'	3/16"
.285"	0.0'	5/16"
.420"	-2.0'	7/16"
.425"	-4.0'	3/8"
.355"	-6.0'	5/16"
.340"	-8.0'	5/16"
.295"	-10.0'	1/4"
.255"	-15.0'	1/4"
.310"	-20.0'	5/16"
Inaccessible	El. -26.0'	Inaccessible

JOB 438-80 NETC, NE VPORT, RI

SHEET NO 11 OF 14

CALCULATED BY ____ DATE ____

CHECKED BY ____ DATE ____

SCALE ____

CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

H-PILE

STEEL THICKNESS MEASUREMENTS

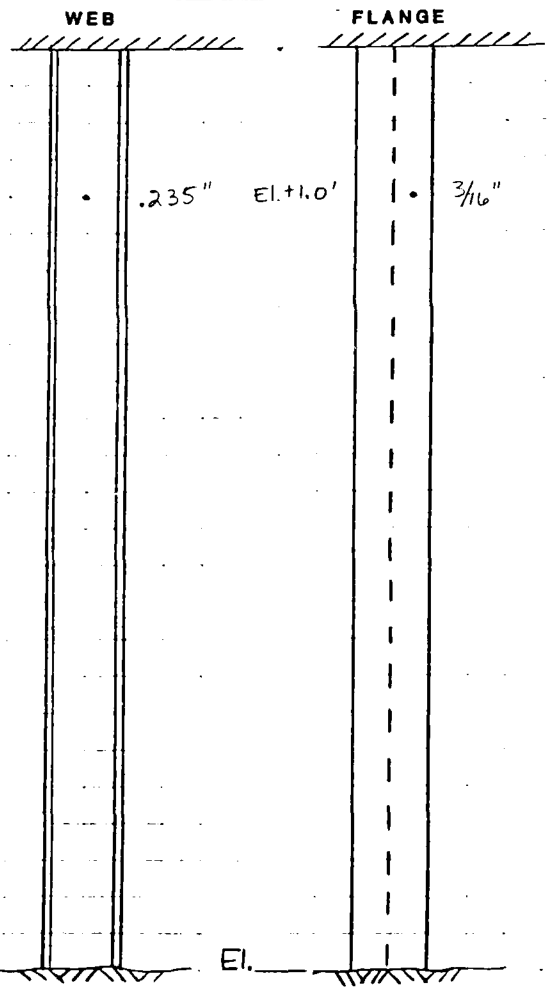
LOCATION: QUAYWALL APRON

BENT 60 PILE B

PILE TYPE HP 14 x 73

ORIGINAL THICKNESS: .506" (WEB)

ORIGINAL THICKNESS: .506" (FLANGE)

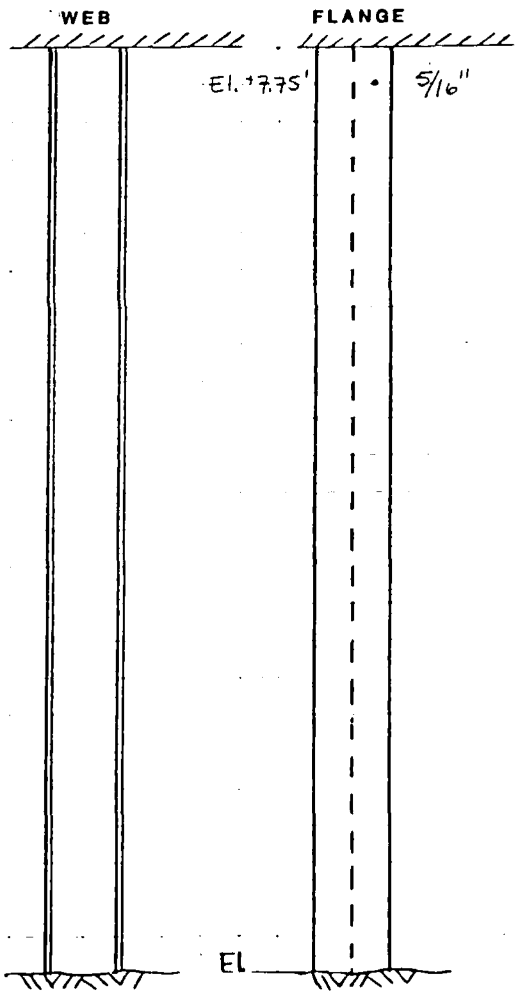


BENT 61 PILE C

PILE TYPE HP 12 x 53

ORIGINAL THICKNESS: .436" (WEB)

ORIGINAL THICKNESS: .436" (FLANGE)



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB 438-80 NETC, NEWPORT, RI

SHEET NO 12 OF 14

CALCULATED BY ______ DATE ______

CHECKED BY ______ DATE ______

SCALE ______

H-PILE

STEEL THICKNESS MEASUREMENTS

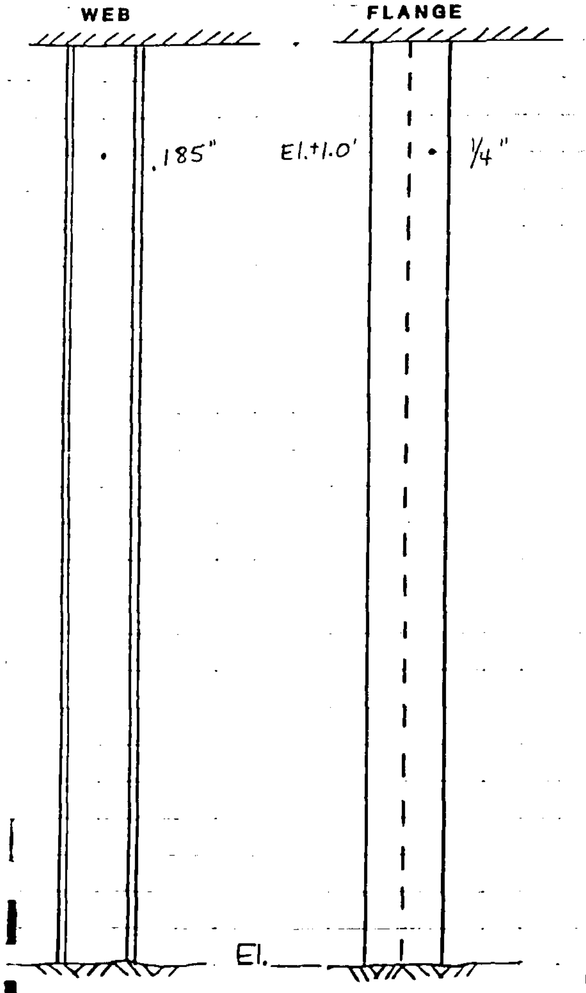
LOCATION Quaywall Apron

BENT 63 PILE C

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE

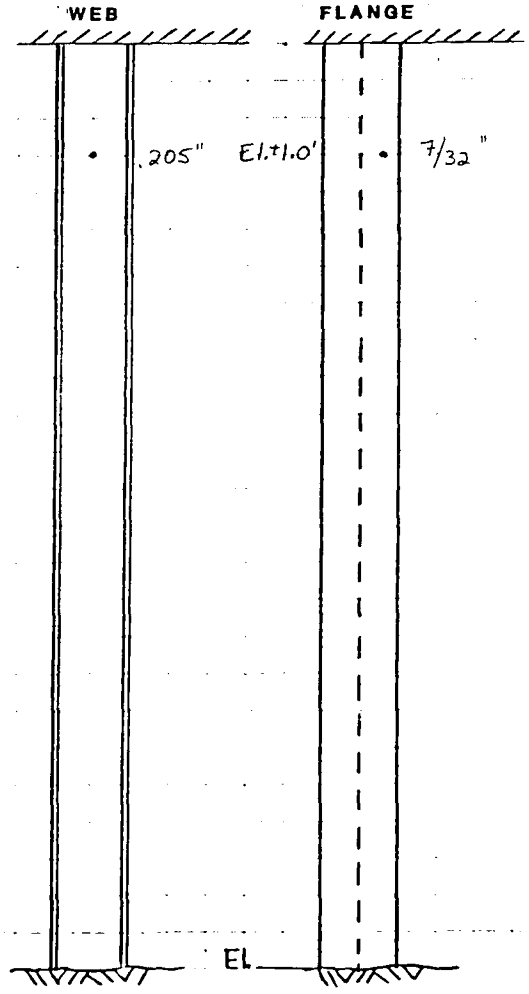


BENT 66 PILE D

PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE



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JOB 438-80 NETC, NEWPORT, RI

SHEET NO 12 OF 14

CALCULATED BY ____ DATE ____

CHECKED BY ____ DATE ____

SCALE ____

H-PILE

STEEL THICKNESS MEASUREMENTS

LOCATION Quaywall Apron

BENT 69 PILE E
PILE TYPE HP 12x53

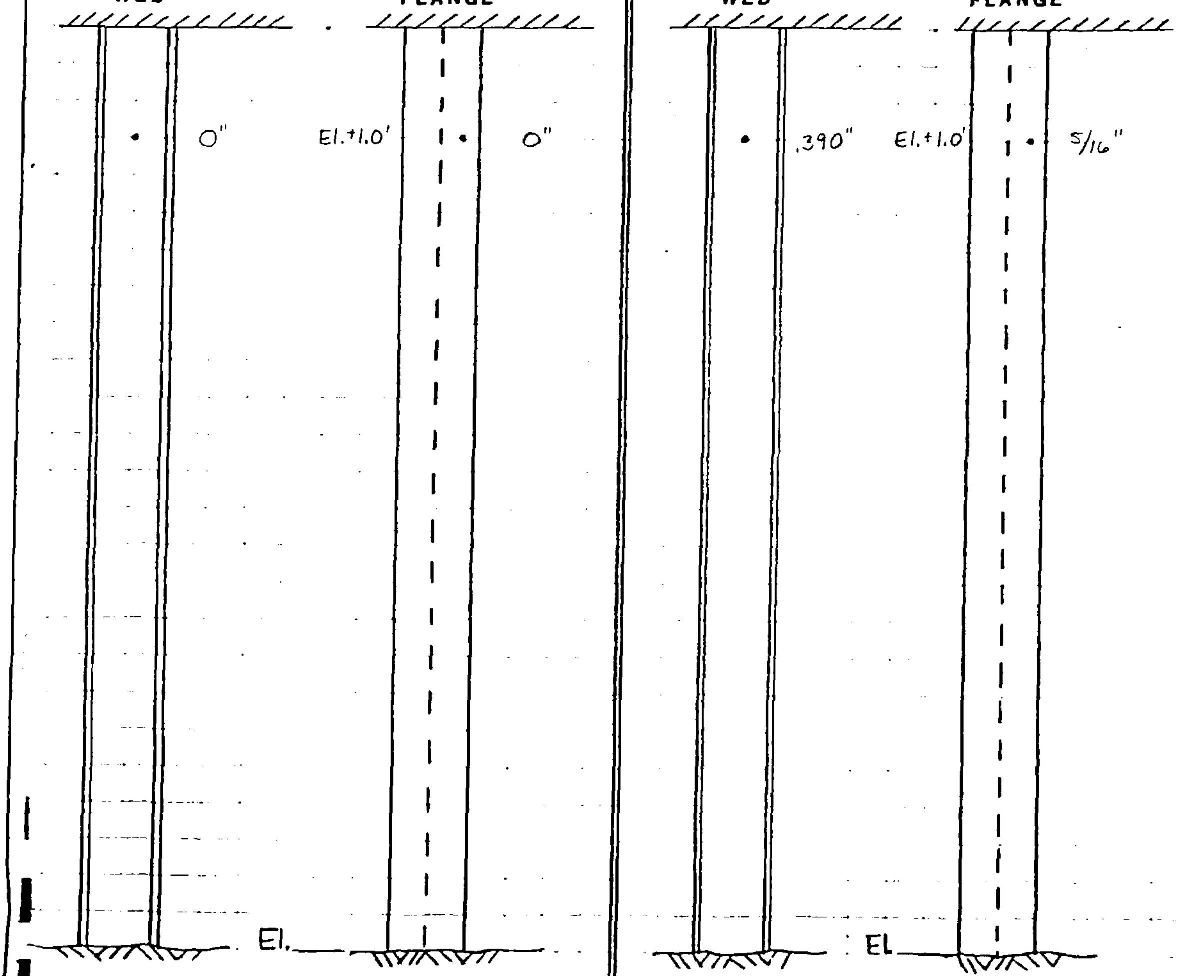
ORIGINAL THICKNESS: .436"
WEB

ORIGINAL THICKNESS: .436"
FLANGE

BENT 72 PILE D
PILE TYPE HP 12x53

ORIGINAL THICKNESS: .436"
WEB

ORIGINAL THICKNESS: .436"
FLANGE



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB 438-80 NETC, NEWPORT, RI

SHEET NO 14 OF 14

CALCULATED BY ____ DATE ____

CHECKED BY ____ DATE ____

SCALE ____

H-PILE

STEEL THICKNESS MEASUREMENTS

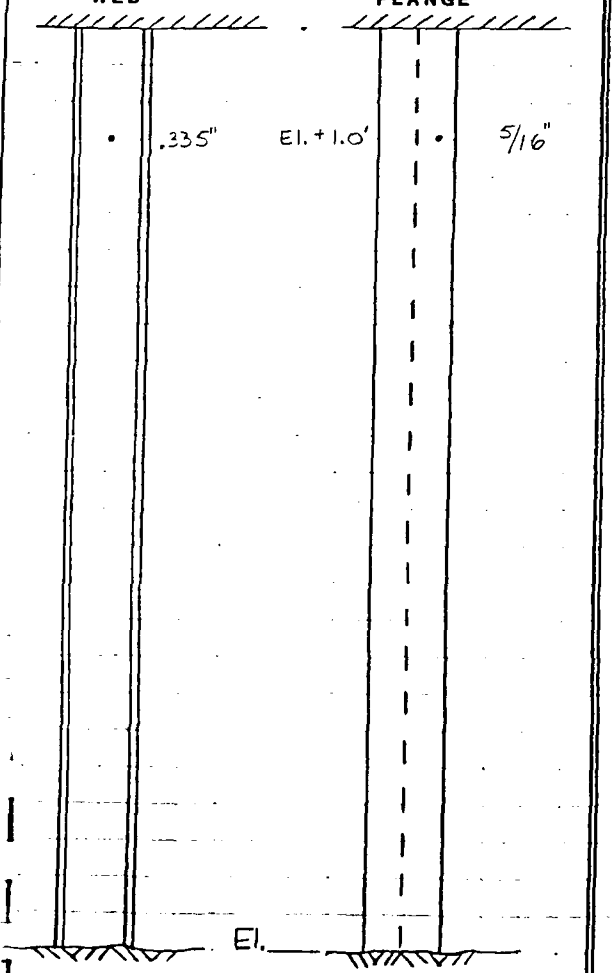
LOCATION Quaywall Apron

BENT 75 PILE C

PILE TYPE HP 12 x 53

ORIGINAL THICKNESS: .436" WEB

ORIGINAL THICKNESS: .436" FLANGE

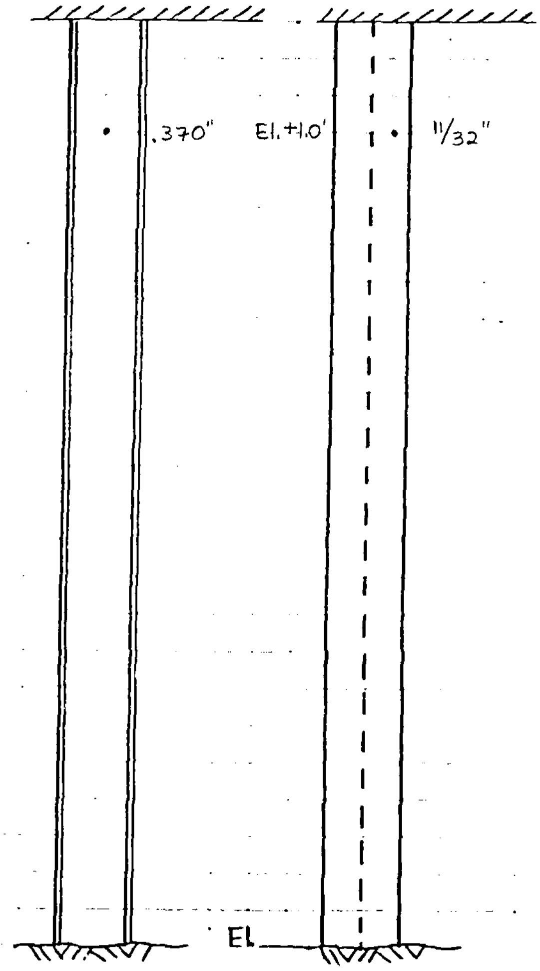


BENT 78 PILE B

PILE TYPE HP 14 x 73

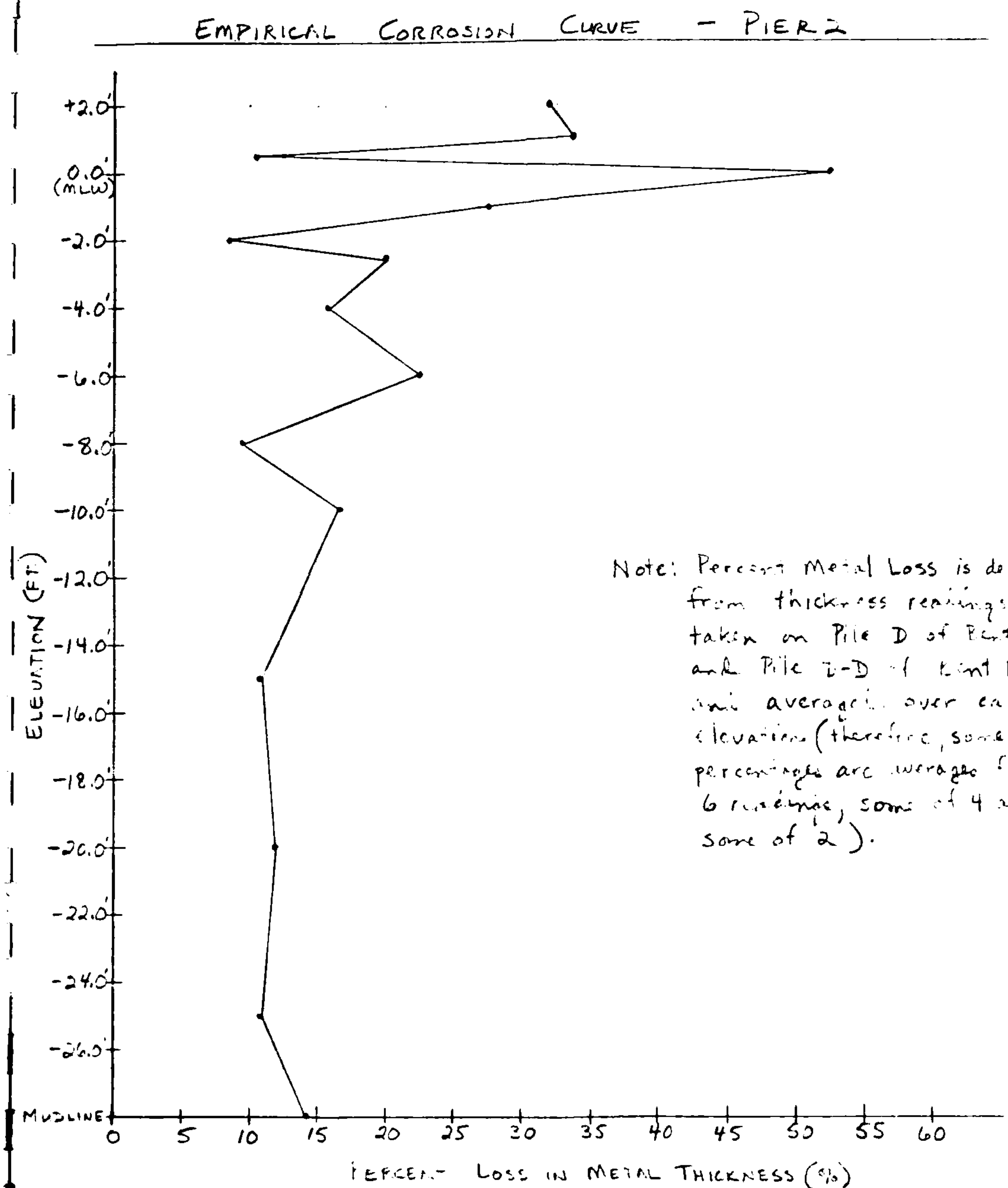
ORIGINAL THICKNESS: .506" WEB

ORIGINAL THICKNESS: .506" FLANGE



CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB 438-80 NETC Newport, RI - PIER 2
SHEET NO 1 OF 1
CALCULATED BY EWL DATE 11-2-81
CHECKED BY DATE
SCALE

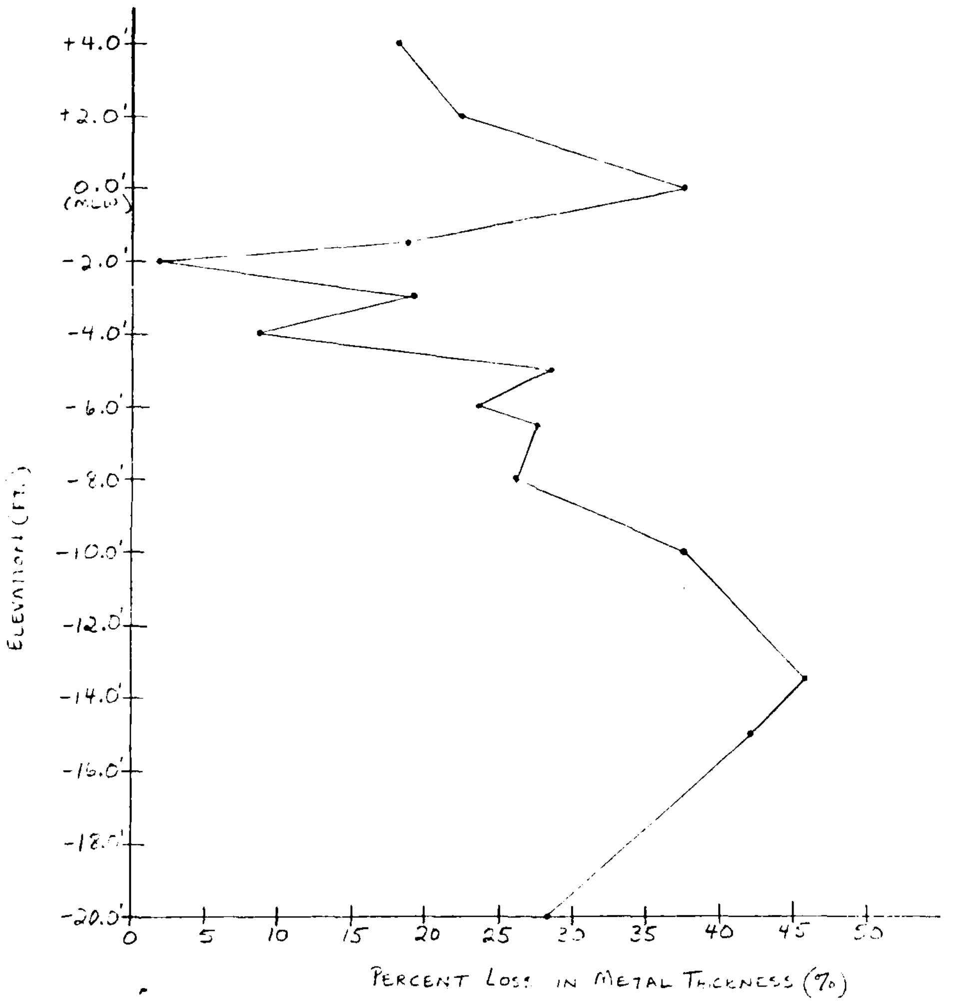


Note: Percent Metal Loss is derived from thickness readings taken on Pile D of Bent 38 and Pile I-D of Bent 113 and averaged over each elevation (therefore, some percentages are averages of 6 readings, some of 4 and some of 2).

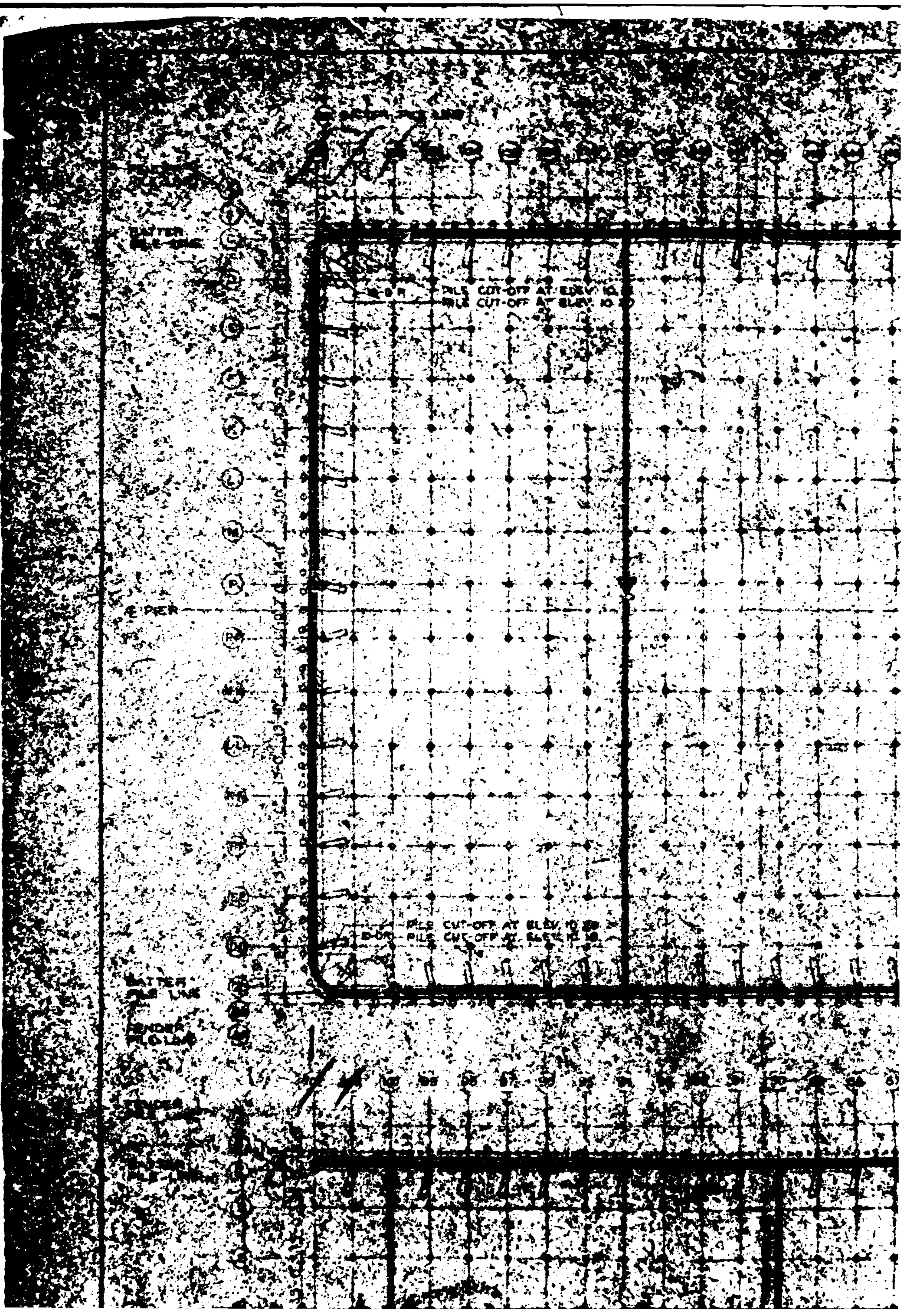
CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

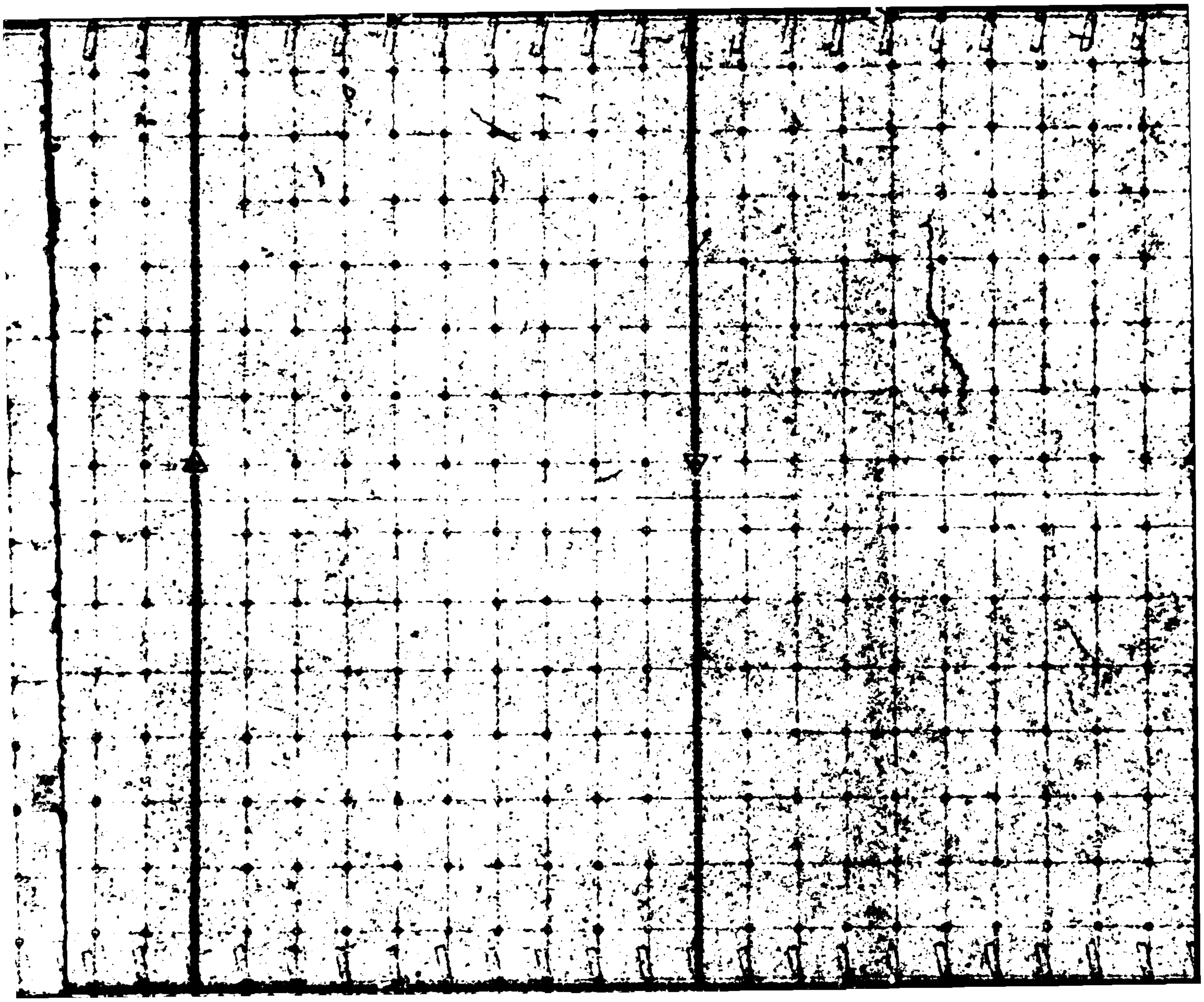
JOB 422-80 NETC Newport, RI - Apron
SHEET NO. 1 OF 1
CALCULATED BY E.J.L. DATE 11-2-81
CHECKED BY DATE
SCALE

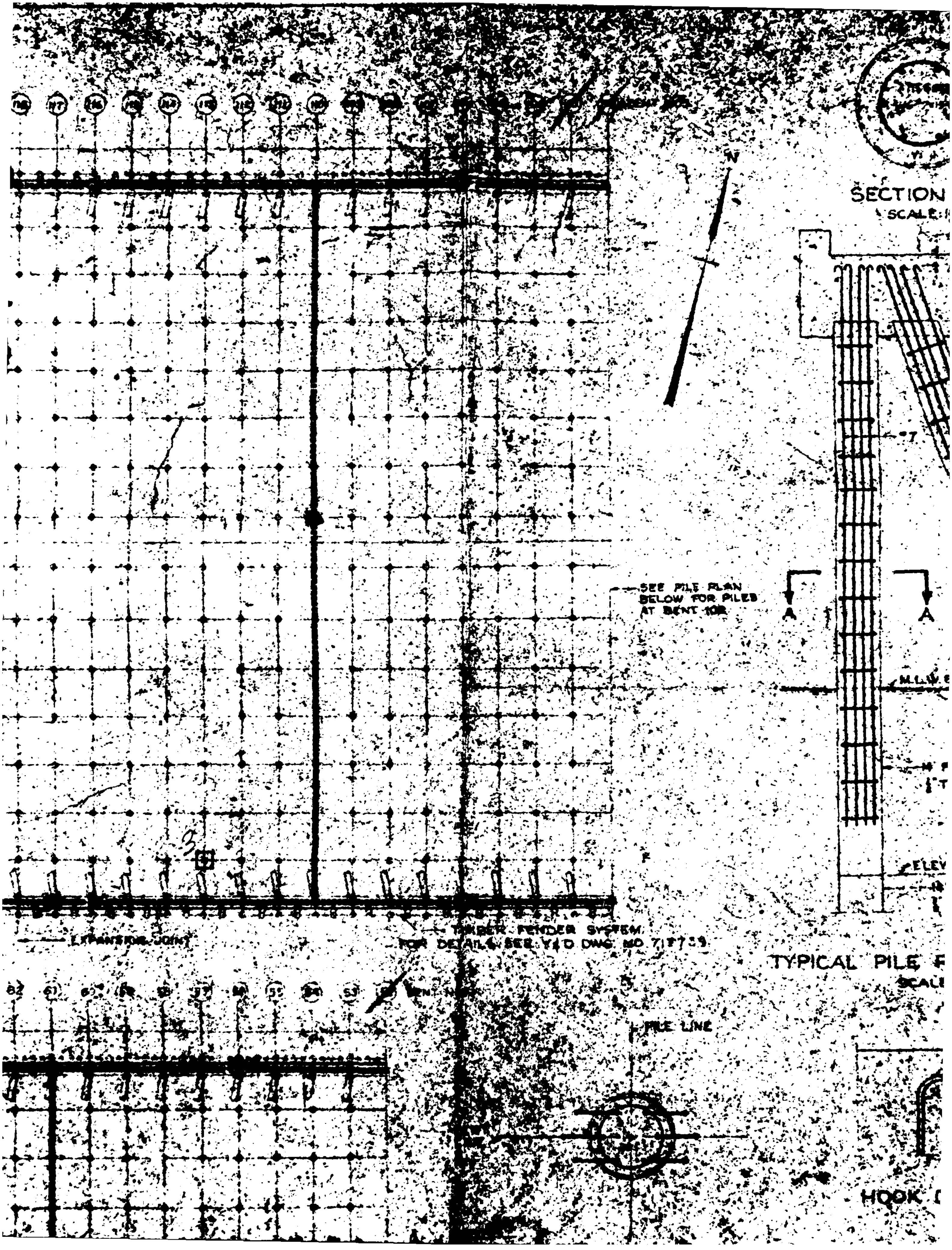
EMPIRICAL CORROSION CURVE – QUAYWALL APRON



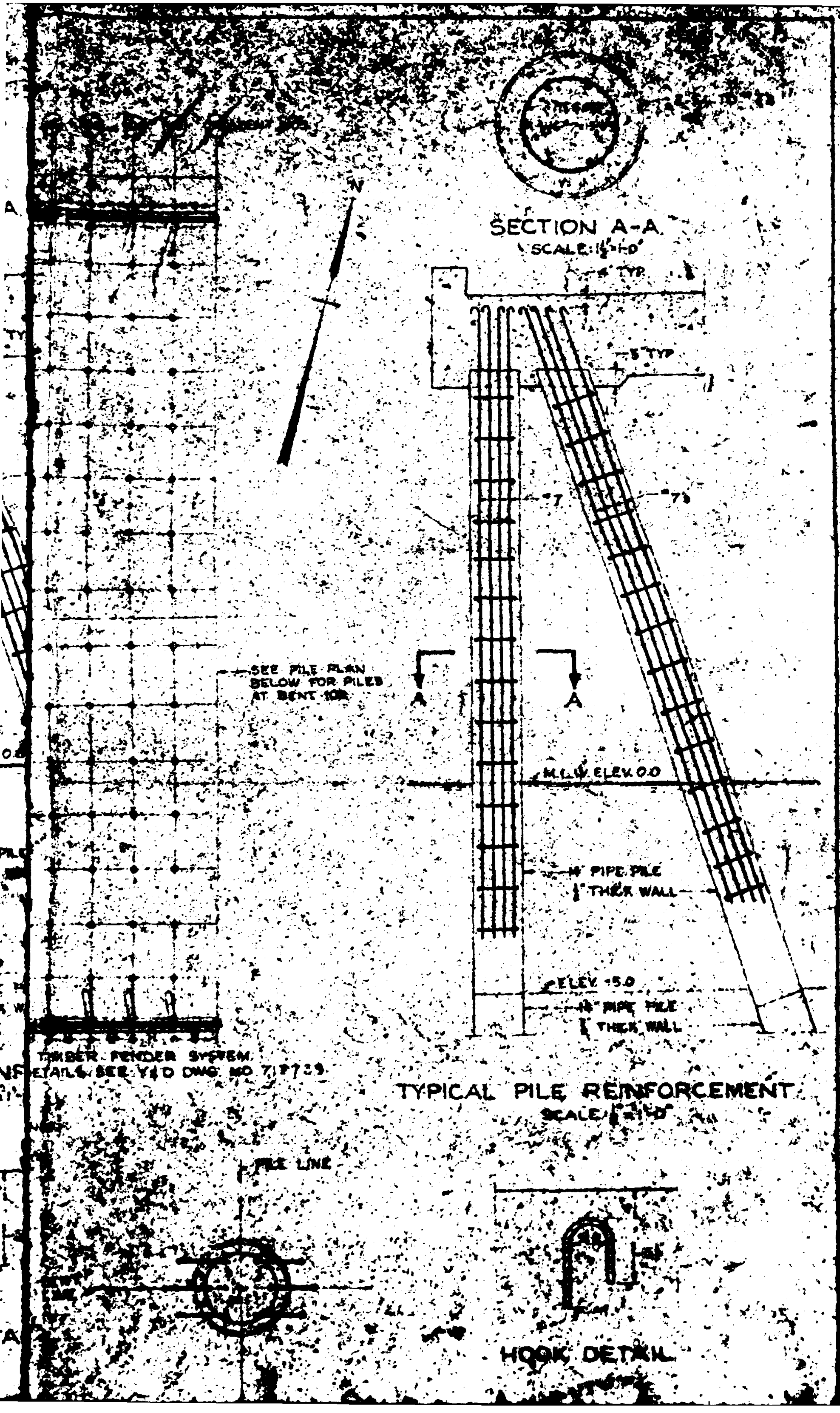
Note: Percent Metal Loss is derived from thickness readings taken on Pile D of Bent 33 and Pile C of Bent 57 and averaged over each elevation (ie, some percentages are averages of 2 readings (web + flange on one pile) and some are averages of 4 readings).


BATTER
PILE LINE
PIER
FENDER
PLG LINE

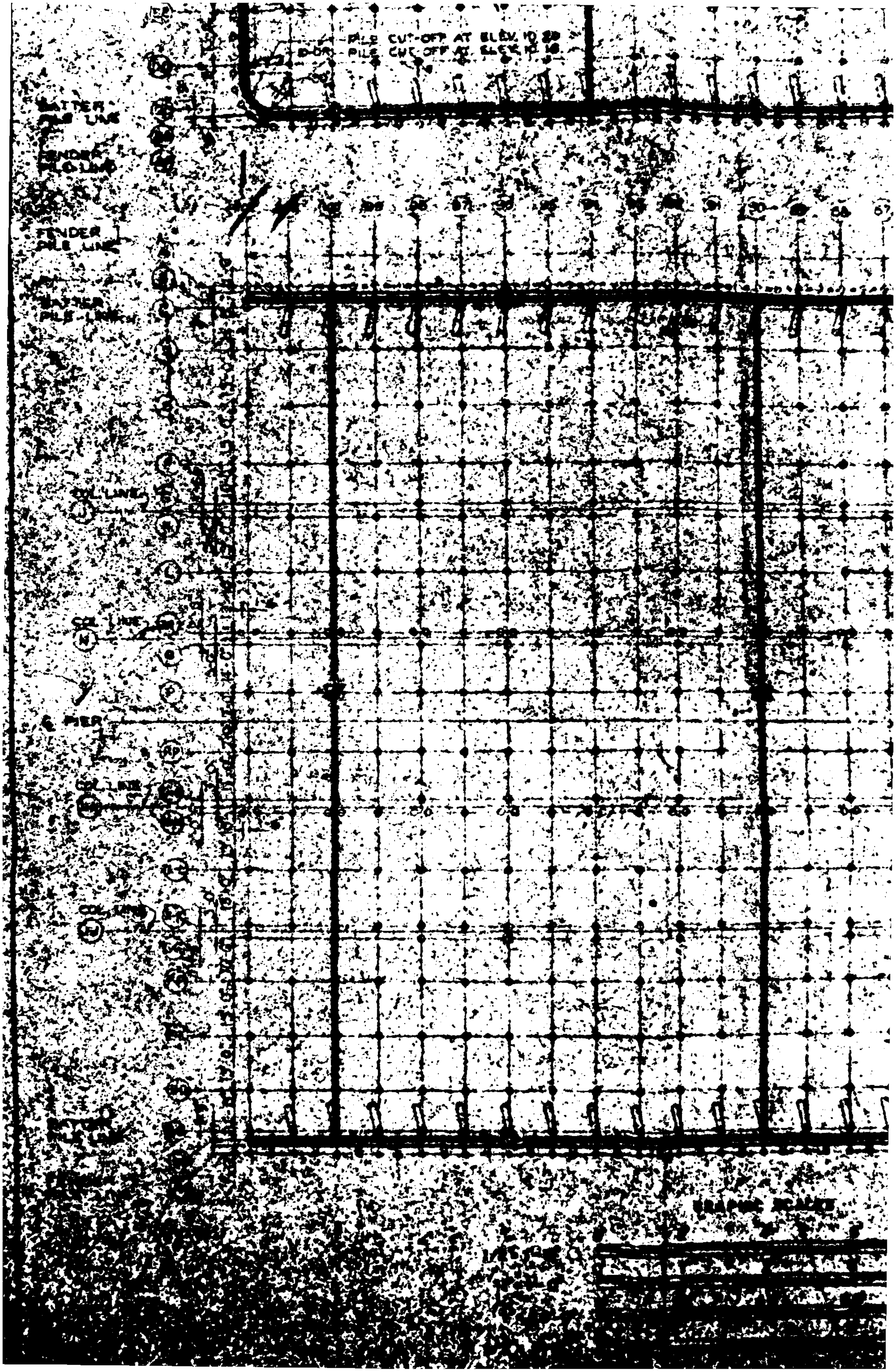


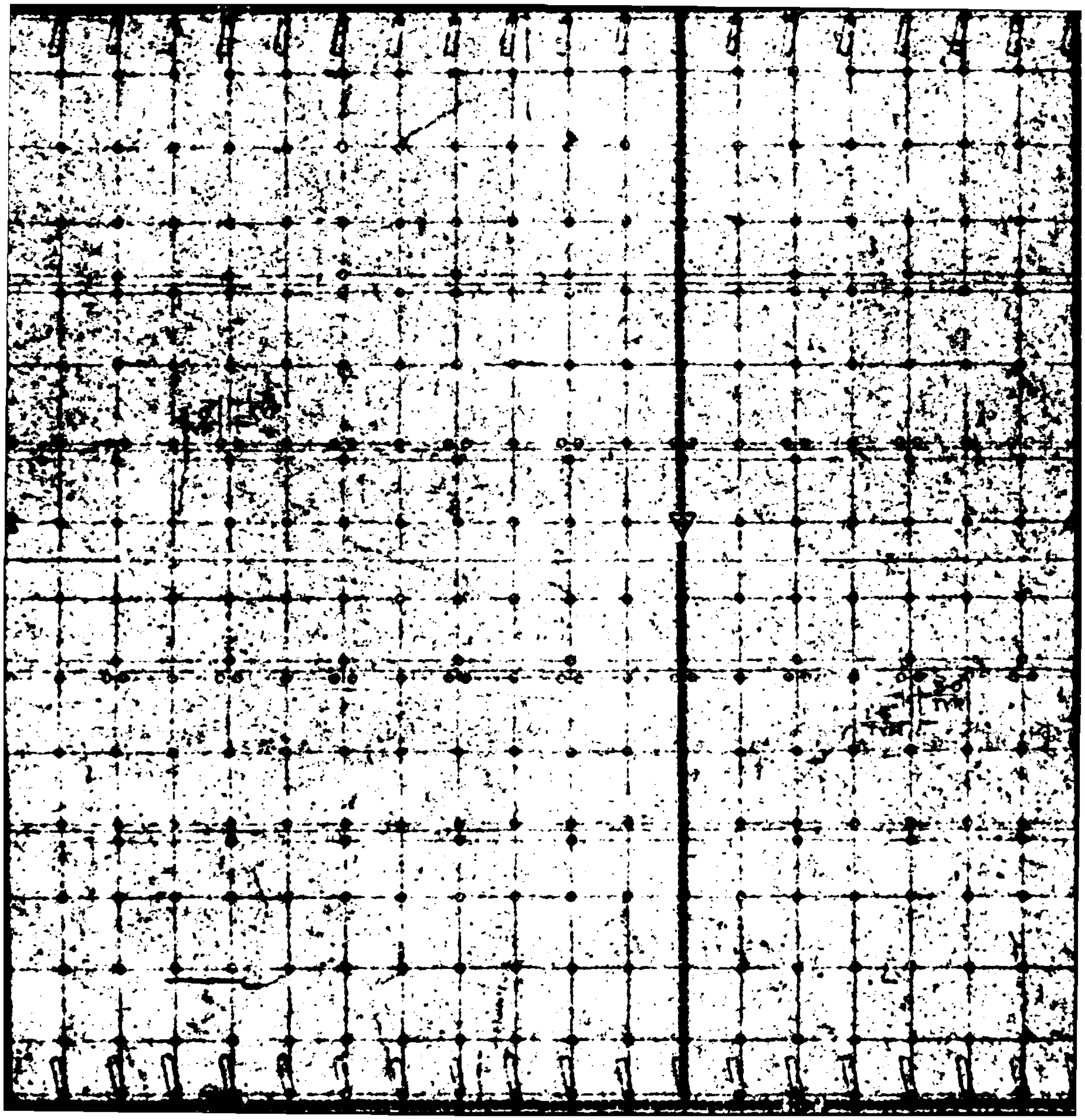


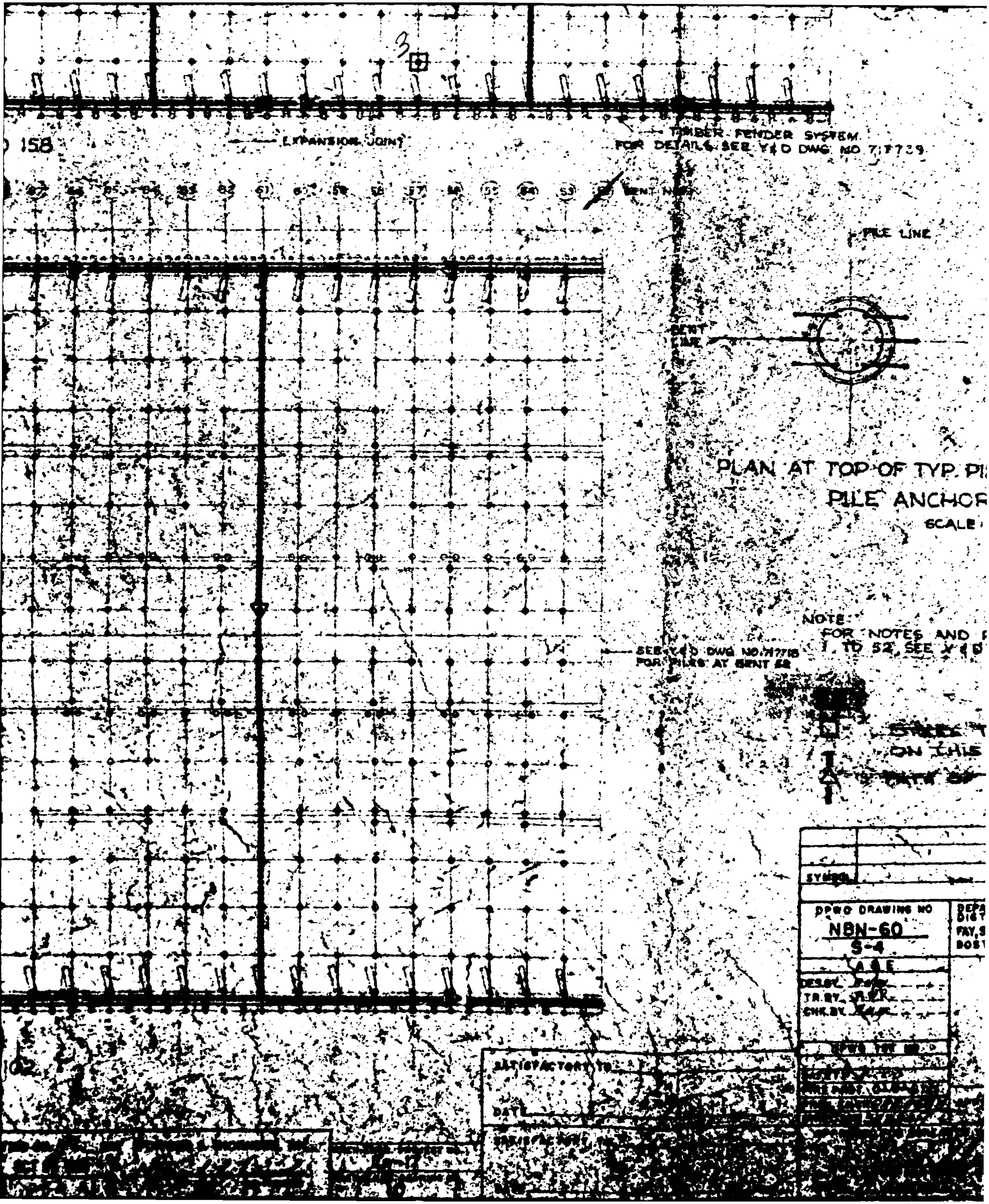
SECTION
SEE PILE PLAN BELOW FOR PILES AT BENT
A
A
EXPANSION JOINT
TIMBER FENDER SYSTEM.
FOR DETAILS SEE Y&D DWG. NO.
TYPICAL PILE
PILE LINE
HOOK



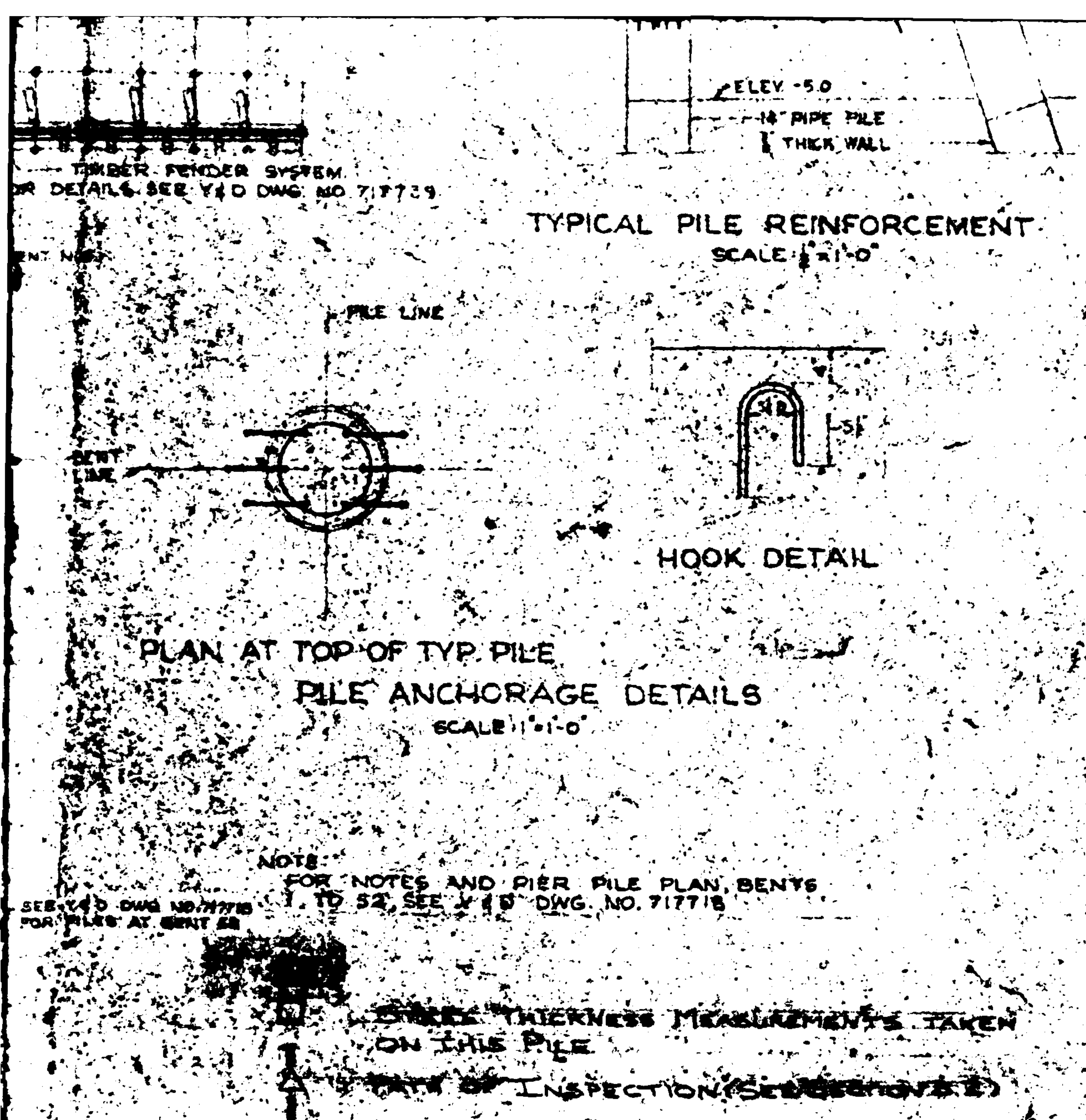
SECTION A-A
SCALE: 1½"=1'-0"
TYPICAL PILE REINFORCEMENT
SEE PILE PLAN BELOW FOR PILES AT BENT 10B
M.L.W. ELEV. 0.0
PIPE PILE
THICK WALL
ELEV. -5.0
PIPE PILE
THICK WALL
TIMBER FENDER SYSTEM
PILE LINE
HOOK DETAIL


PILE CUT-OFF AT ELEV.
PILE CUT-OFF AT ELEV.
FENDER
PILE LINE
COL. LINE
COL. LINE
COL. LINE





3
158
EXPANSION JOINT
TIMBER FENDER SYSTEM
FOR DETAILS SEE Y&D DWG NO 717739
BENT NO
PILE LINE
BENT LINE
PLAN AT TOP OF TYP. PI
PILE ANCHOR
SCALE
NOTE:
FOR NOTES AND
1 TO 52, SEE Y&D
SEE Y&D DWG NO 717718
FOR PILES AT BENT 52
ON THIS
SYMBOL
DPWO DRAWING NO
NBN-60
S-4
DES. BY
TR. BY
CHK. BY
SATISFACTORY TO
DATE
162

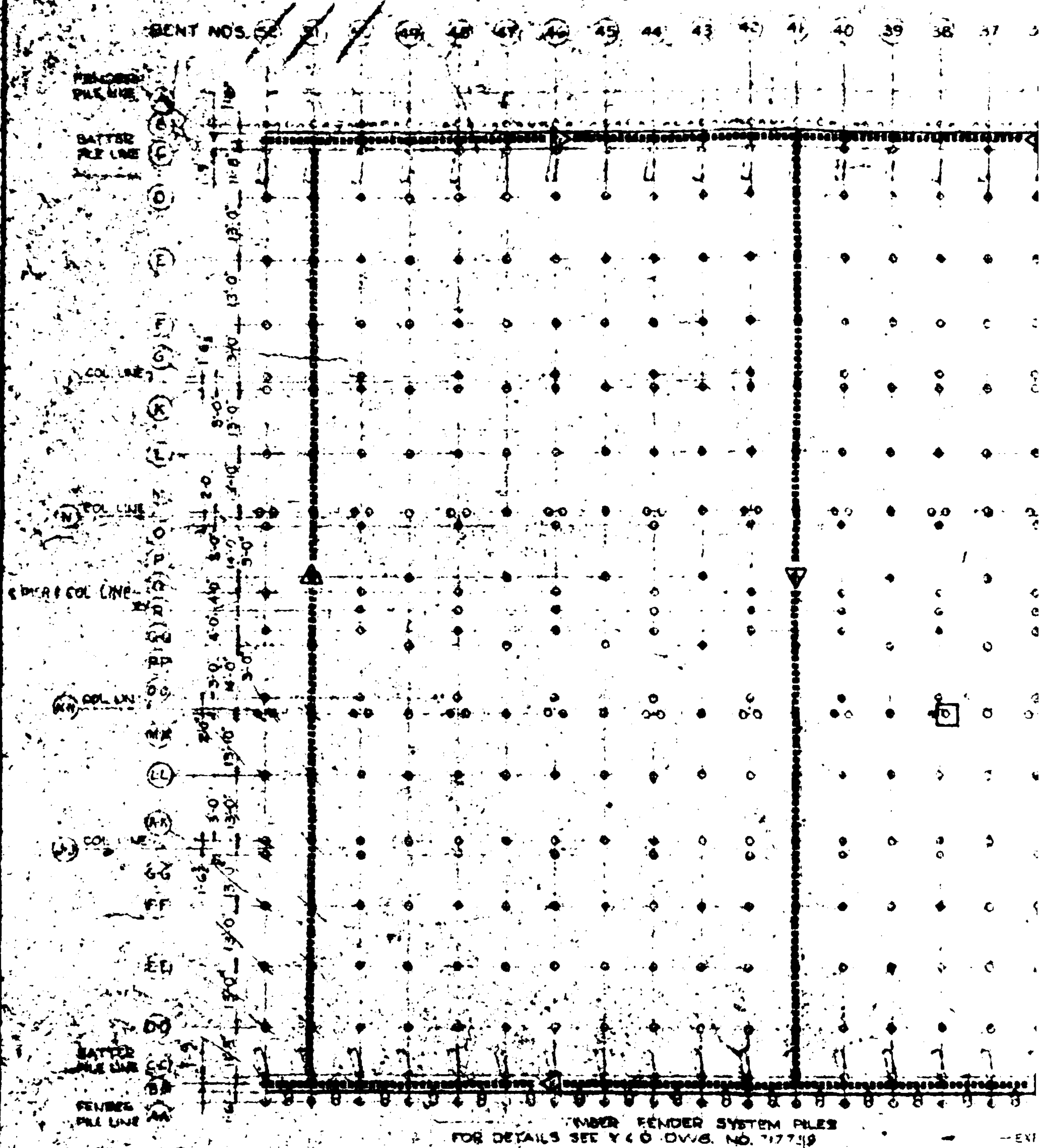


SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

DPWO DRAWING NO. NBN-60 S-4	DEPARTMENT OF THE NAVY, DISTRICT PUBLIC WORKS OFFICE — BUREAU OF YARDS & DOCKS 1st ND BOSTON, MASS. FAY, SPOFFORD & THORNDIKE, INC. - CHARLES A. MAGUIRE & ASSOCIATES BOSTON, MASS ARCHITECT - ENGINEER PROVIDENCE, R.I.
A&E DESGN. TR. BY. CHK. BY.	U. S. NAVAL BASE NEWPORT, R. I. **PIER NO. 2, BULKHEAD AND UTILITIES** **PILE PLAN - BENTS 52-158**

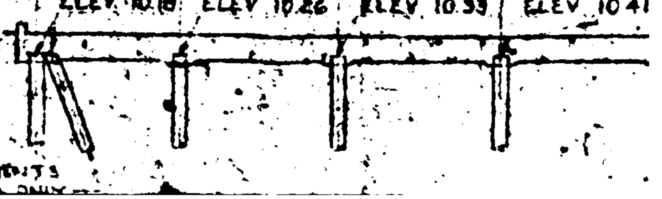
717718



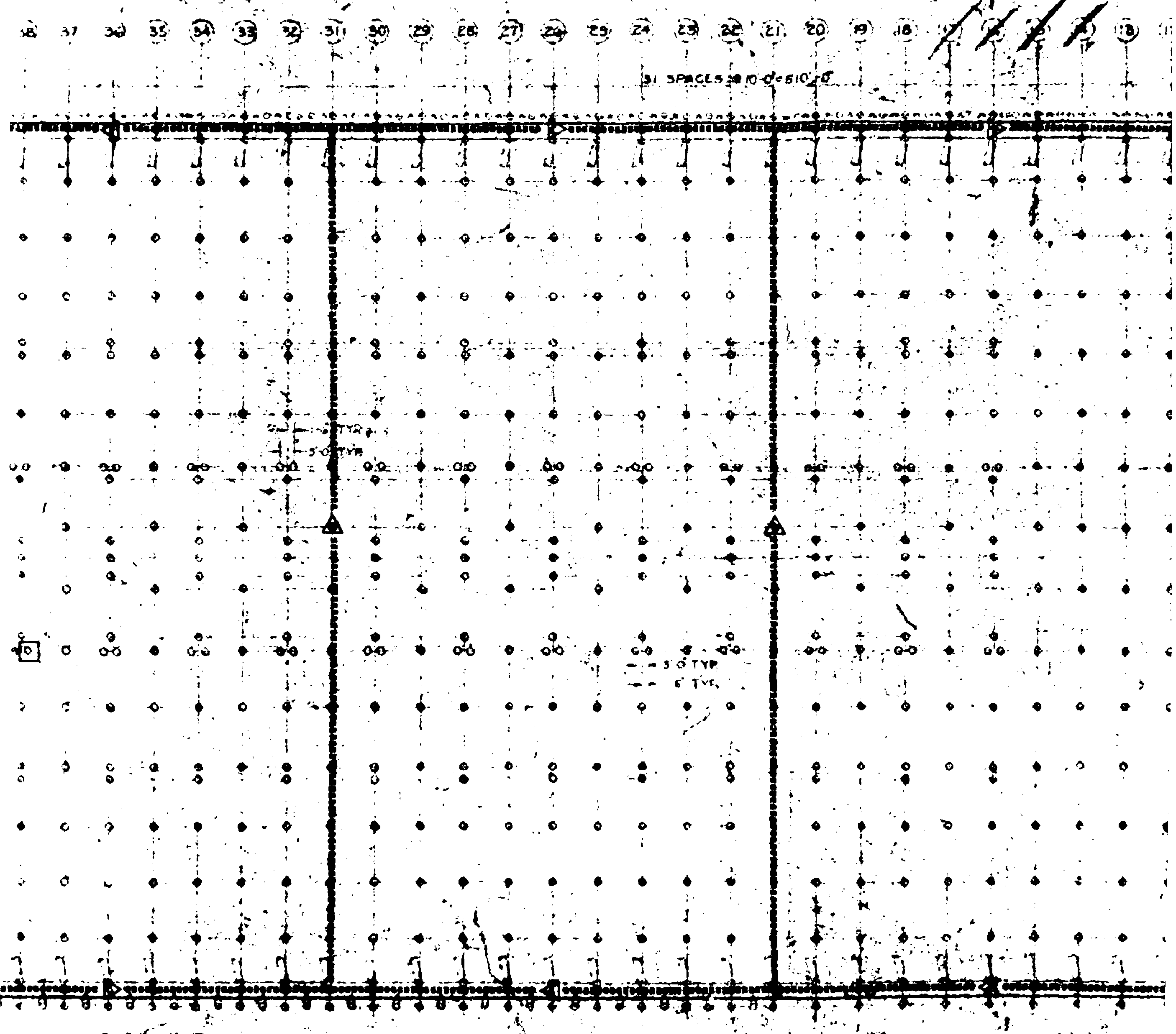
TIMBER FENDER SYSTEM PILES
FOR DETAILS SEE Y & D DWG. NO. 7177?9

	BB	DD	EE	FF
BENT 1	ELEV. 10.60	ELEV. 10.60	ELEV. 10.60	ELEV. 10.60
BENT 2	ELEV. 10.50	ELEV. 10.50	ELEV. 10.50	ELEV. 10.50
BENT 3	ELEV. 10.40	ELEV. 10.40	ELEV. 10.40	ELEV. 10.40
BENT 4	ELEV. 10.31	ELEV. 10.31	ELEV. 10.30	ELEV. 10.37
BENT 5	ELEV. 10.21	ELEV. 10.24	ELEV. 10.30	ELEV. 10.37
BENTS 103 & 15	ELEV. 10.18	ELEV. 10.26	ELEV. 10.33	ELEV. 10.41

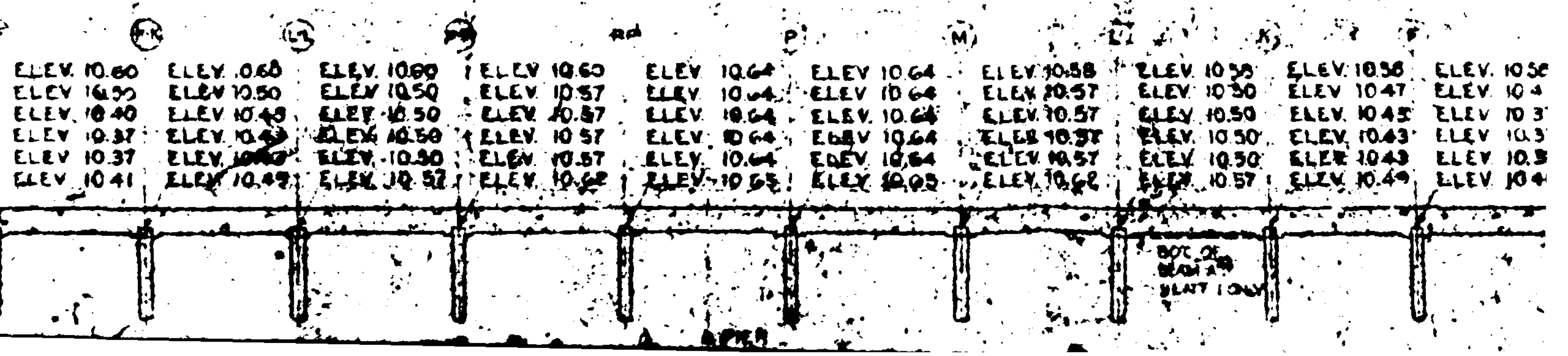
CURB AT BENTS 15 & 103 ONLY

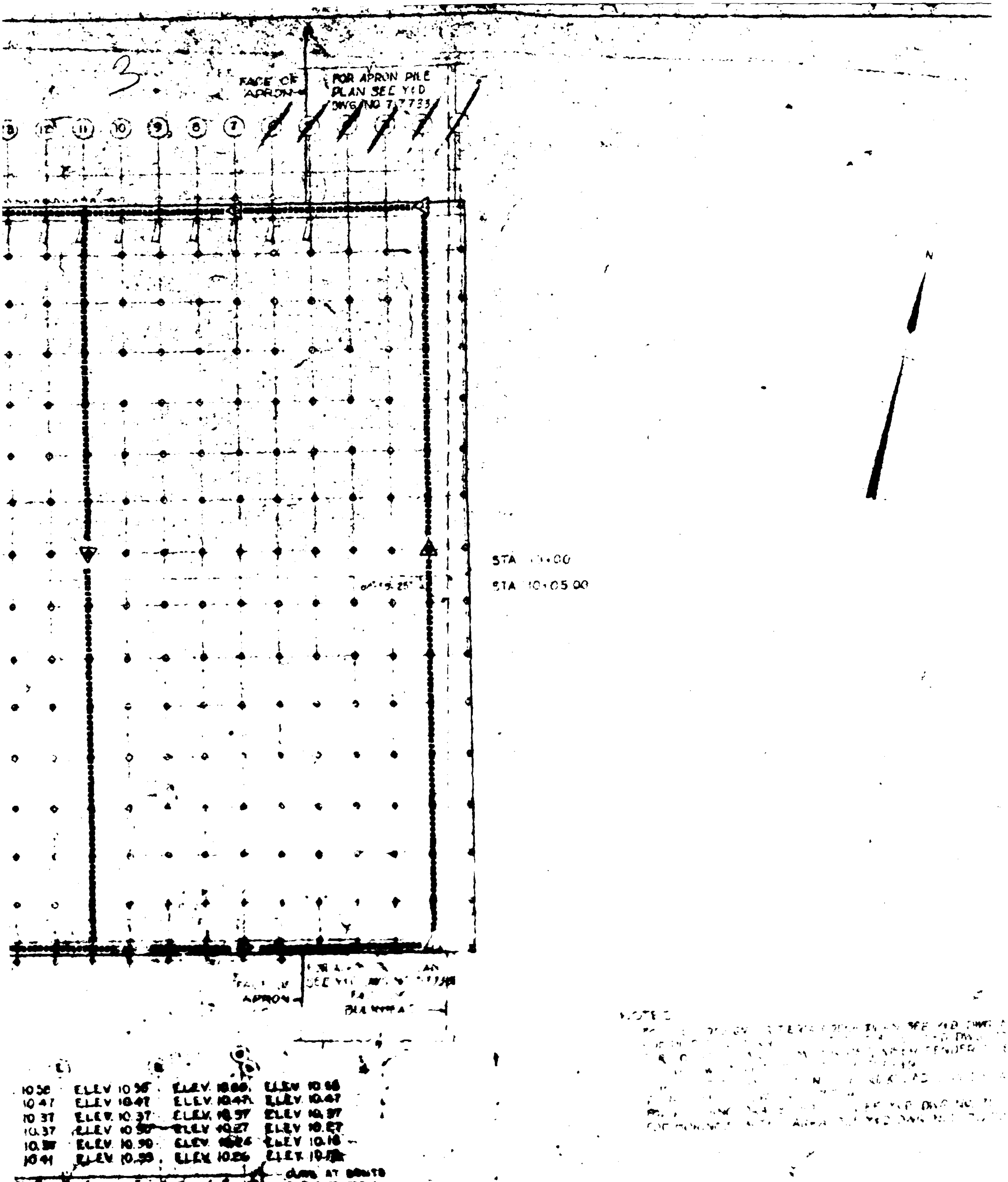


PILE AT BENTS 5 & 103 ONLY



PILE PLAN - PIER 2 - BENT 1 TO 52
SCALE 1" = 20'-0"

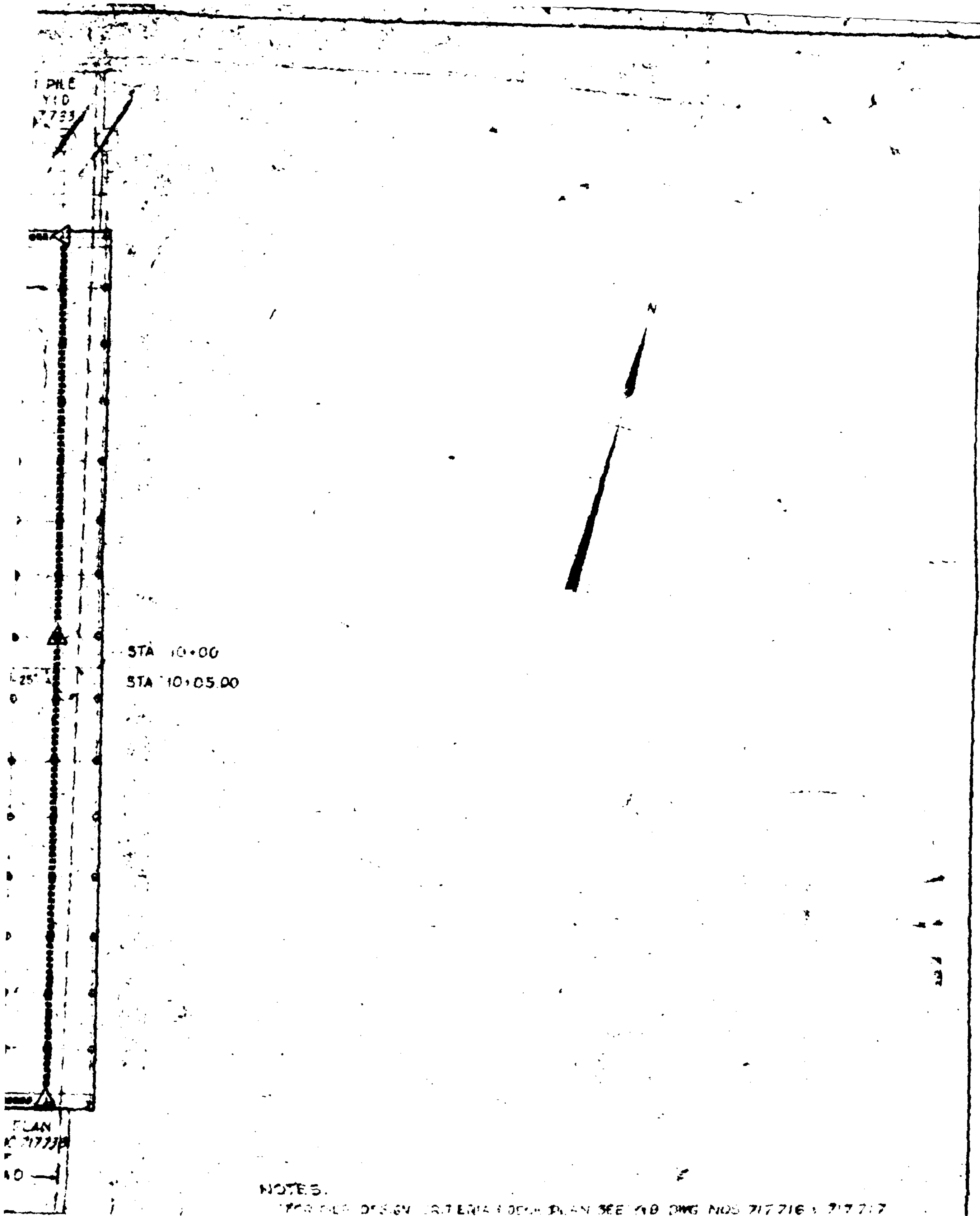




KEY

☐ - STEEL THICKNESS MEASURE

4

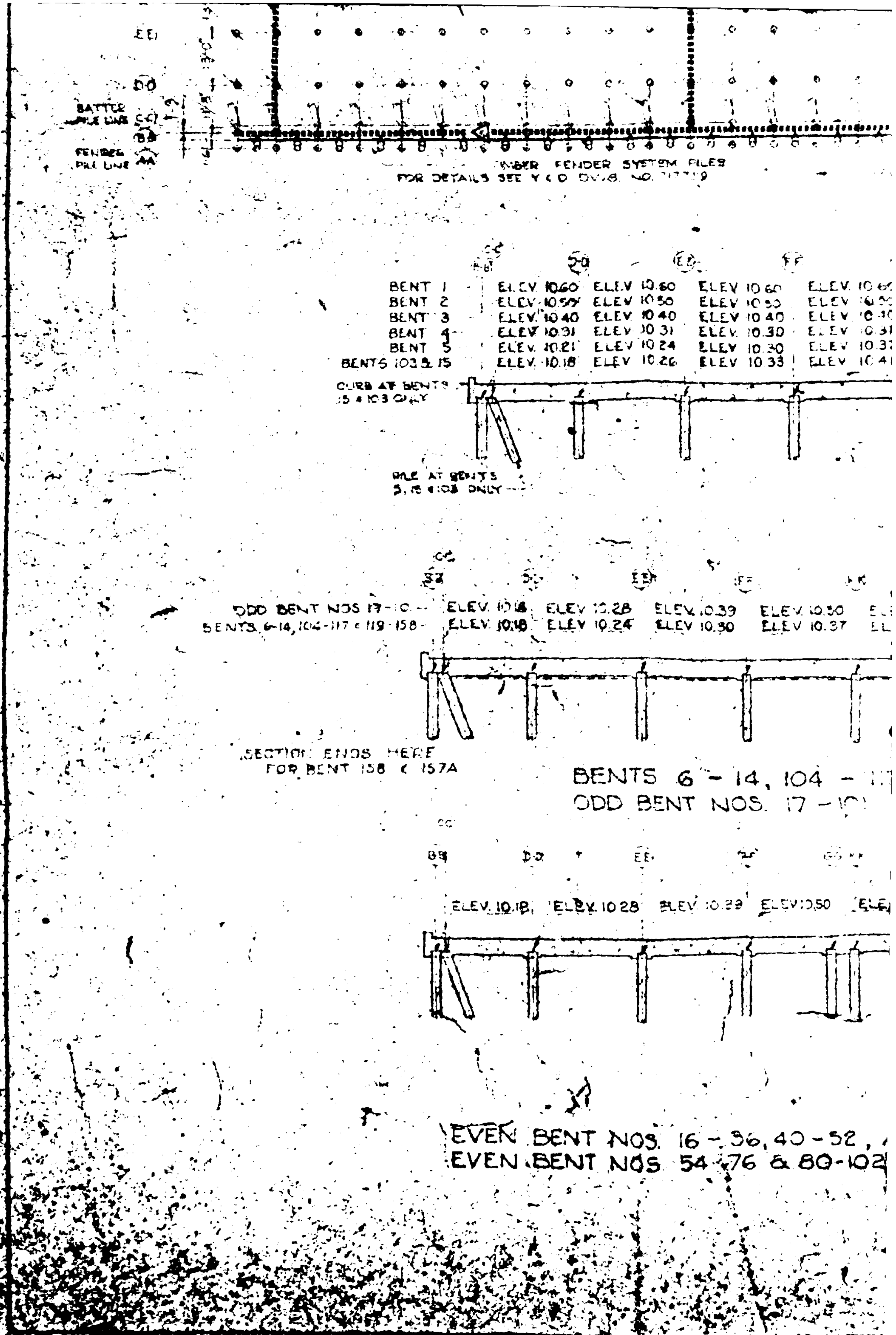


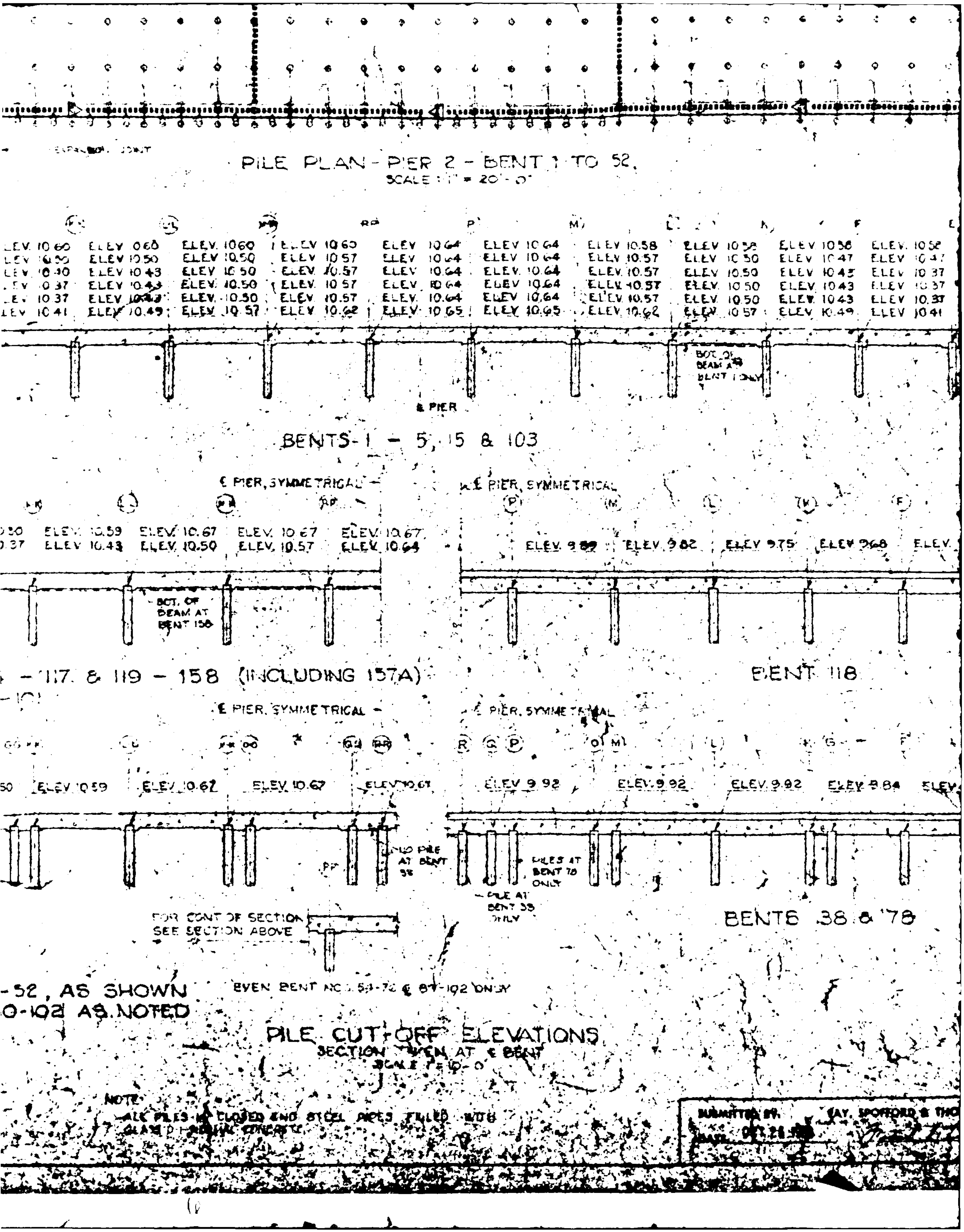
NOTES.

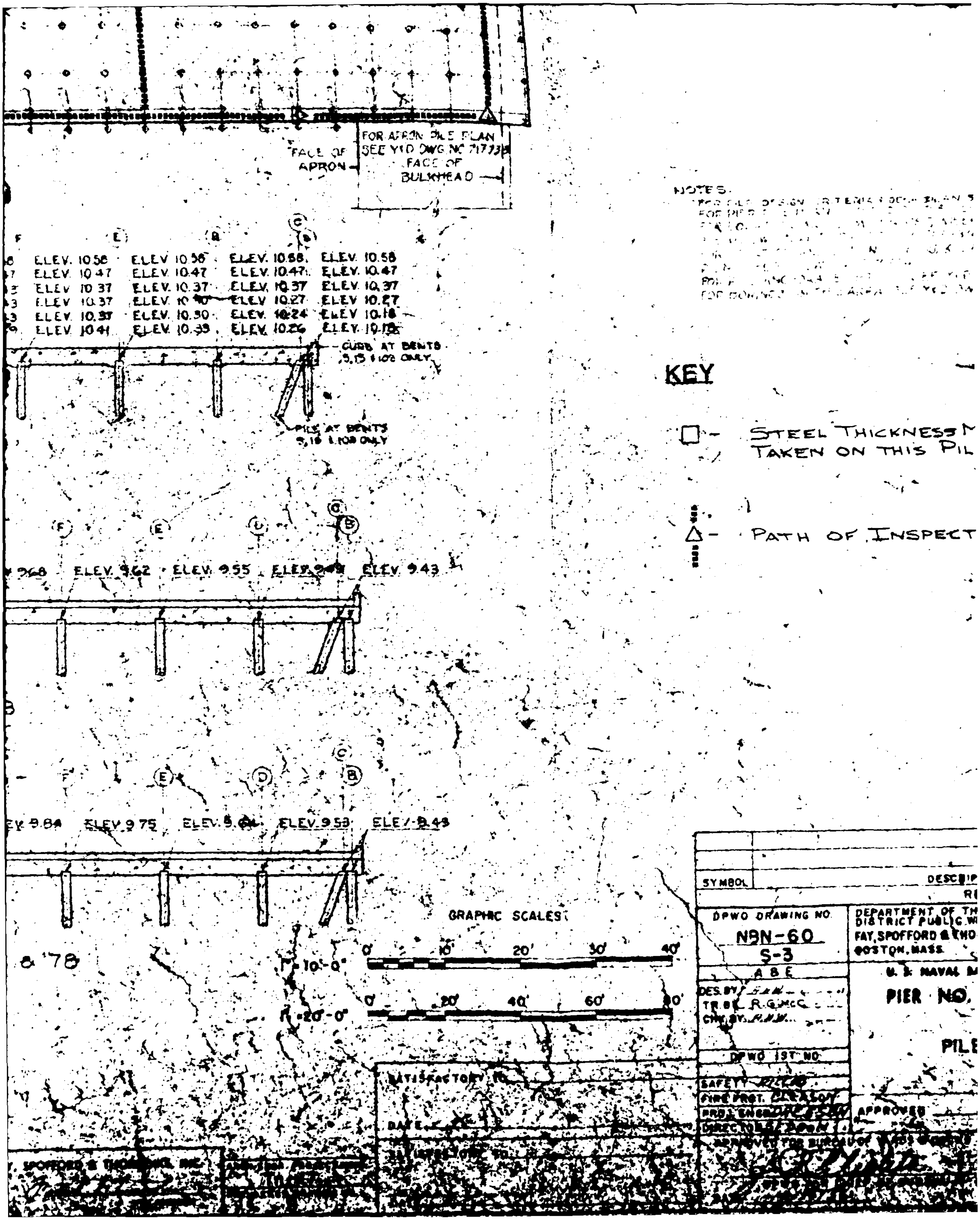
FOR PIER DESIGN CRITERIA & DECK PLAN SEE YD DWG NOS 717716 & 717717
FOR PIER [illegible] DWG [illegible]
FOR LOCATION [illegible] TIMBER FENDER PILES
[illegible]
[illegible] AND SHEET
[illegible]
[illegible] SEE YD DWG NO. [illegible]
FOR BORINGS IN THIS AREA SEE YD DWG NOS [illegible]

KEY

☐ = STEEL THICKNESS MEASUREMENTS


EE
DD
BATTER PILE LINE CC
BB
FENDER PILE LINE AA
TIMBER FENDER SYSTEM PILES
FOR DETAILS SEE Y & D DWG. NO.
BB
DD
EE
FF
BENT 1 ELEV 10.60 ELEV 10.60 ELEV 10.60
BENT 2 ELEV 10.50 ELEV 10.50
BENT 3 ELEV 10.40 ELEV 10.40 ELEV 10.40
BENT 4 ELEV 10.31 ELEV 10.31 ELEV 10.30
BENT 5 ELEV 10.21 ELEV 10.24 ELEV 10.30
BENTS 103 & 15 ELEV 10.18 ELEV 10.26 ELEV 10.33
CURB AT BENTS 15 & 103 ONLY
PILE AT BENTS 5, 15 & 103 ONLY
ODD BENT NOS 17 - ELEV 10.18 ELEV 10.28 ELEV 10.39 ELEV 10.50
BENTS 6-14, 104-117 & 119-158 - ELEV 10.18 ELEV 10.24 ELEV 10.30 ELEV 10.37
SECTION ENDS HERE
FOR BENT 158 & 157A
BENTS 6 - 14, 104 -
ODD BENT NOS. 17 -
BB
DD
EE
ELEV 10.18 ELEV 10.28 ELEV 10.39 ELEV 10.50
EVEN BENT NOS. 16 - 36, 40 - 52,
EVEN BENT NOS 54 - 76 & 80-102


EXPANSION JOINT
PILE PLAN - PIER 2 - BENT 1 TO 52.
SCALE 1" = 20'-0"
ELEV. 10.60 ELEV. 10.60 ELEV. 10.60 ELEV. 10.63 ELEV. 10.64 ELEV. 10.64 ELEV. 10.58 ELEV. 10.58 ELEV. 10.58 ELEV. 10.52
ELEV. 10.50 ELEV. 10.50 ELEV. 10.50 ELEV. 10.57 ELEV. 10.64 ELEV. 10.64 ELEV. 10.57 ELEV. 10.50 ELEV. 10.47 ELEV. 10.47
ELEV. 10.40 ELEV. 10.43 ELEV. 10.50 ELEV. 10.57 ELEV. 10.64 ELEV. 10.64 ELEV. 10.57 ELEV. 10.50 ELEV. 10.43 ELEV. 10.37
ELEV. 10.37 ELEV. 10.43 ELEV. 10.50 ELEV. 10.57 ELEV. 10.64 ELEV. 10.64 ELEV. 10.57 ELEV. 10.50 ELEV. 10.43 ELEV. 10.37
ELEV. 10.37 ELEV. 10.43 ELEV. 10.50 ELEV. 10.57 ELEV. 10.64 ELEV. 10.64 ELEV. 10.57 ELEV. 10.50 ELEV. 10.43 ELEV. 10.37
ELEV. 10.41 ELEV. 10.49 ELEV. 10.57 ELEV. 10.62 ELEV. 10.65 ELEV. 10.65 ELEV. 10.62 ELEV. 10.57 ELEV. 10.49 ELEV. 10.41
BOT. OF BEAM AT BENT 1 ONLY
℄ PIER
BENTS 1 - 5, 15 & 103
℄ PIER, SYMMETRICAL
ELEV. 10.59 ELEV. 10.67 ELEV. 10.67 ELEV. 10.67
ELEV. 10.43 ELEV. 10.50 ELEV. 10.57 ELEV. 10.64
BOT. OF BEAM AT BENT 158
- 117, & 119 - 158 (INCLUDING 157A)
℄ PIER, SYMMETRICAL
ELEV. 9.89 ELEV. 9.82 ELEV. 9.75 ELEV. 9.68 ELEV.
BENT 118
℄ PIER, SYMMETRICAL
ELEV. 10.59 ELEV. 10.62 ELEV. 10.67 ELEV. 10.67
NO PILE AT BENT 38
℄ PIER, SYMMETRICAL
ELEV. 9.92 ELEV. 9.92 ELEV. 9.92 ELEV. 9.84 ELEV.
PILES AT BENT 78 ONLY
PILE AT BENT 38 ONLY
BENTS 38 & 78
FOR CONT. OF SECTION SEE SECTION ABOVE
EVEN BENT NO. 53-72 & 87-102 ONLY
-52, AS SHOWN
0-102 AS NOTED
PILE CUT-OFF ELEVATIONS
SECTION TAKEN AT ℄ BENT
SCALE 1" = 10'-0"
NOTE:
ALL PILES IN CLOSED END STEEL PIPES FILLED WITH CLASS ... CONCRETE.
SUBMITTED BY:
FAY, SPOFFORD & THO...
DATE: OCT. 28 ...



FOR APRON PILE PLAN SEE Y&D DWG NO 717738
FACE OF APRON
FACE OF BULKHEAD
NOTES:
ELEV. 10.58 ELEV. 10.58 ELEV. 10.58 ELEV. 10.58
ELEV. 10.47 ELEV. 10.47 ELEV. 10.47 ELEV. 10.47
ELEV. 10.37 ELEV. 10.37 ELEV. 10.37 ELEV. 10.37
ELEV. 10.37 ELEV. 10.27 ELEV. 10.27
ELEV. 10.30 ELEV. 10.24 ELEV. 10.18
ELEV. 10.41 ELEV. 10.35 ELEV. 10.26
CURB AT BENTS 5, 15 & 102 ONLY
PILE AT BENTS 5, 15 & 102 ONLY
KEY
STEEL THICKNESS TAKEN ON THIS PIL
PATH OF INSPECT
ELEV. 9.62 ELEV. 9.55 ELEV. 9.43
ELEV. 9.75 ELEV. 9.53
GRAPHIC SCALES
0' 10' 20' 30' 40'
1" = 10'-0"
0' 20' 40' 60' 80'
1" = 20'-0"
SYMBOL
DPWO DRAWING NO.
NBN-60
S-3
DEPARTMENT OF TH
DISTRICT PUBLIC W
FAY, SPOFFORD & THO
BOSTON, MASS.
U. S. NAVAL
PIER NO.
PILE
DES. BY
TR. BY R.G.McC
CHK. BY
DPWO 1ST NO.
SAFETY
FIRE PROT.
PROJ. ENGR.
DIRECTOR
APPROVED
SATISFACTORY TO
DATE
SPOFFORD & THORNDIKE, INC.

PILE PLAN
DWG NO 717735
[illegible] OF
BULKHEAD

NOTES.

FOR [illegible] DESIGN [illegible] SEE Y&D DWG NO. 717716 & [illegible]
FOR PIER [illegible] Y&D DWG [illegible]
FOR LO[illegible] TIMBER FENDER PILES
[illegible]
FOR [illegible] BULKHEAD [illegible] AND SHEET
[illegible]
FOR [illegible] SEE Y&D DWG NO. [illegible]
FOR BORINGS IN THIS AREA SEE Y&D DWG NOS. 717703 & 717704

KEY

□ - STEEL THICKNESS MEASUREMENTS TAKEN ON THIS PILE

△ - PATH OF INSPECTION (SEE SECTION 3.2)

GRAPHIC SCALES

10' 20' 30' 40'

20' 40' 60' 80'

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

DPWO DRAWING NO.
NBN-60
S-3
A&E

DES. BY [illegible]
TR. BY R.G.McC.
CHK. BY R.W.W.

DPWO 1ST NO.
SAFETY [illegible]
FIRE PROT. [illegible]

DEPARTMENT OF THE NAVY
DISTRICT PUBLIC WORKS OFFICE
BUREAU OF YARDS & DOCKS
1st ND BOSTON, MASS.

FAY, SPOFFORD & THORNDIKE, INC. - CHARLES A. MAGUIRE & ASSOCIATES
BOSTON, MASS. ARCHITECT - ENGINEER PROVIDENCE, R.I.

U.S. NAVAL BASE NEWPORT, R.I.

PIER NO. 2, BULKHEAD AND UTILITIES

PILE PLAN - BENTS 1-52

APPROVED

8

END
DATE
FILMED
6 - 86
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